In presenting this dissertation as a partial fulfillment of the requirements for an advanced degree from the Georgia Institute of Technology, I agree that the Library of the Institute shall make it available for inspection and circulation in accordance with its regulations governing materials of this type.

I agree that permission to copy from, or to publish from, this dissertation may be granted by the professor under whose direction it was written, or, in his absence, by the Dean of the Graduate Division when such copying or publication is solely for scholarly purposes and does not involve potential financial gain.

It is understood that any copying from, or publication of, this dissertation which involves potential financial gain will not be allowed without written permission.

Richard Thomas Douty

CHARACTERISTICS OF FLEXIBLE FLANGE CONNECTIONS AND FASTENERS

A THESIS

Presented to

the Faculty of the Graduate Division

by

Richard Thomas Douty

In Partial Fulfillment

of the Requirements for the Degree

Master of Science in Civil Engineering

Georgia Institute of Technology

June, 1957

3 ~~~
~~~

CHARACTERISTICS OF FLEXIBLE FLANGE

CONNECTIONS AND FASTENERS

Approved:

Dr. F. W. Schutz, Jr.

Austin B. Caseman

Dr. Karl M. Murphy

Date Approved by Chairman: May 28, 1957
~~~

ACKNOWLEDGMENTS

The writer wishes to thank all those whose advice and generosity contributed to the successful completion of this thesis.

To Dr. F. W. Schutz go my warmest thanks for the invaluable guidance and encouragement which he generously extended throughout the program. To Professor A. B. Caseman and Dr. Karl M. Murphy go thanks for the many helpful comments and suggestions incorporated in its written result.

I would like to extend my appreciation to the following companies for material which they generously donated:

The Atlantic Steel Company
Steel, Incorporated
Calvert Iron Works, Incorporated
Russell, Burdsall and Ward Bolt and Nut Company.

Thanks are also due Mr. C. M. Pavey of the Civil Engineering Department machine shop whose efforts made possible the fabrication of the testing equipment for this study.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS

t = thickness of flange

t_w = thickness of tension web

M_w = moment at the web of a Tee section

M_b = moment at the bolts of a Tee section

M_p = plastic moment

k = coefficient for correction of ultimate moment due to area reduction; equals A_n/A_g

A_n = area of cross-section minus holes

A_g = gross area of cross-section

e = edge distance

d = distance from center of bolts to edge of web

b = length of cross-section undergoing bending

σ_y = yield stress

P = tensile force on section causing flange bending

P_u = ultimate tensile force causing flange bending

P'_u = ultimate prying force

ΣT = summation of tensile forces in a bolt group

T_i = bolt pretension

T_y = tension in a bolt at first yield of the outer fibers of the flange

T_u = tension in a bolt at flange ultimate

W = width of flange resisting compression on bracket

y = height of compression area of bracket

m = distance from bottom of compression area to first fastener row of the bracket

T = tension in a bolt

n = number of rows of bolts in moment bracket

S = section modulus of gross section

G = gage

x = eccentricity of the bracket load

F = friction force resisting slip

P_{bu} = ultimate bracket load

SUMMARY

CHARACTERISTICS OF FLEXIBLE FLANGE CONNECTIONS AND FASTENERS

The investigation had as its objectives the determination of the characteristics of a connection employing a flexible flange and a study of the additional load placed on the fasteners as a result of prying action. A rational approach has been developed for the analysis and equations have been derived that can be used in the design of such flexible connections. The method used employs the principle of simple plastic theory.

Experimental work was used to verify the theoretical development of the subject. This work was divided into two phases. The first phase was devised to check the correlation between the analysis and the test results. The correlation was checked by subjecting Tee sections to axial tension loads along the web and studying the bending characteristics of the flange and the tension in the fasteners through the full range of loading. It was found that the equations developed could be accurately used to compute the failure load of a flexible connection with given dimensions, and furthermore, that the tension in the fasteners could be accurately predicted at any load, including loads beyond computed ultimate. "Ultimate" here is defined as that load at which excessive deformations occur. Above the ultimate load strain hardening resists the external load.

The second phase of testing involved the fabrication of a moment bracket and testing it to observe the accuracy with which the developed

analysis described its behavior. It was found that the analysis could well serve as a means for design. The bracket failed when expected as far as flexibility of the flange was concerned and exhibited a good amount of reserve strength due to strain hardening. Corollary observations were made of the slip characteristics of the flange and shearing forces in the web.

In the working load range with a safety factor of two against ultimate occurring both series of tests exhibited elastic deflection characteristics.

Since test results of the Tee sections and moment bracket essentially verified the theoretical approach to the subject a typical design of a Tee stub moment connection was presented but not tested.

As the testing involved a variety of severe conditions, it can be considered that the developed design approach is applicable to most flexible flange connections where either flexibility or a definite knowledge of the failure load is desired.

CHAPTER I

INTRODUCTION

The collapse method of design, commonly called plastic design in this country, depends upon a structural steel member attaining a specified moment at certain critical points along its span. As the degree of loading increases and certain of these points become plastified the moment there becomes constant, the magnitude depending upon the yield characteristics and the geometry of the beam section. A further increase in load causes other regions of high moment to become plastified while the original regions of plastification undergo rotation, though still maintaining the plastic moment. When enough of these hinges have formed to cause a mechanism, the member will have reached its ultimate load carrying capacity. The design load of this member then will be this ultimate load divided by an appropriate safety factor. By this method each member of a structural system and the entire system can be designed with a uniform safety factor so that greater economy results.

The object of this investigation was to use these ultimate concepts in carrying out a study having the following objectives.

1. To determine the behavior of the flanges of a Tee when the stem is subject to tensile loads. Such a condition occurs in a beam-to-column moment connection where stub Tees serve as connections from the upper and lower flanges of the beam to the column.

2. To determine the behavior of the fasteners of such Tees.

3. To develop a design procedure for a moment bracket and investigate the behavior of a typical bracket including slip characteristics.

4. To determine a design procedure for a Tee-stub moment connection.

5. To determine the effect initial pretension has on the ultimate load on the flange and the ultimate load on the bolt.

The nature of the general problem demands that some part of the connections be sufficiently flexible to deform freely under a certain load. A Tee-type connection, where an external load forces the wings of the flange to undergo bending, provides such flexibility if properly dimensioned.

In a moment bracket the capacity of a vertical pattern of connectors is greatest if each connector reaches its yield load and the summation of the resulting tensile forces is balanced by compressive stress acting on a small area at the lower edge of the connection. This can occur under two conditions: first, that the topmost connectors are ductile enough to remain at yield stress while the lower connectors are picking up load, or second, that the connection plate be flexible enough to allow an even distribution of load to each bolt. Each is an ideal state, but a combination of the two could most likely produce the desired result. The connectors, however, are affected by flexibility of the flange as this state produces prying forces in addition to the tensile forces resulting from the external load. If the resultant force in the connectors becomes too large, the connectors will inevitably fail.

At the ultimate load condition the connectors may go into bearing and be loaded in shear. The connector problem then becomes even more critical because of the reduction in ultimate tensile load associated with added shear load.

The experimental investigation was carried out in two phases:

1. An attempt was made to correlate the external load with the flexing action of a connection flange. A study of the forces present in the connectors at all loads was included. The tests were carried out with an assembly that had no shearing stress applied to the connectors.

Simple plastic theory has been used to obtain a logical explanation for the ultimate load behavior of these Tee sections.

2. Once the flange behavior was established, a moment-shear connection was designed with flange flexibility and tested to see if it would carry the predicted ultimate moment and have sufficient rotation capacity so that the methods could be readily adaptable to plastic design concepts.

High tensile strength bolts were used as the connectors because of the inherent effective use made of such bolts by placing them in tension.

CHAPTER II

THEORETICAL DEVELOPMENT

Tee Section

The analysis is based upon the assumption that when the flange of a bolted Tee connection (Fig. 4) undergoes excessive deformation as a result of tensile forces acting on the stem, plastic hinges have occurred at the bolt lines and at the edges of the web. Observation of such a specimen that had been loaded to this extent verified this assumption.

Since a fixed beam with a concentrated load at the center has the same moments at the ends as at the center in the elastic range, the elastic solution of such a case is no different from the plastic solution due to the fact that no redistribution of moments occurs. At the ultimate load the moments described above have become fully plastic.

The free bodies shown in Fig. 5 are associated with the above assumptions. Since the plastic hinges begin forming before the deformation of so short a span is discernable to the eye, it can be assumed that the magnitude of the deformation causes no error in the analysis.

The coefficient "k" allows for the reduction in ultimate moment capacity at the section through the holes. It is merely the ratio of the area through the holes to the area of a solid section.

The free bodies shown in Fig. 5 can be used to compute the theoretical tension in the bolts at any stage of the loading.

When the pretension force is ignored, the predicted curve of bolt tension versus load is divided into three separate segments.

The first segment expresses the force in the bolts as a linear function of the elastic moments at the ends and center of a fixed ended beam with a concentrated load at the center. Since the moments are equal for this condition, "k" equals one. The stresses at the bolt line are higher than at the web throughout the elastic range. The upper limit of this segment is the load at which the steel at the bolt line begins to yield.

The middle segment is a transition curve in which the moment at the bolt holes is becoming plastic while the moment at the web is approaching the plastic condition. The web moment lags the bolt moment in becoming fully plastic. The upper limit of this segment is the load at which both moments have become fully plastic.

The third segment expresses the force in the bolts as a linear function of the moments at the ends and center after each has become plastified and strain hardening commences. The moments will exist in the ratio of "k" to unity because of the geometry of the cross-section. The load at which both moments become completely plastic is called P_u.

The equations below can be used to construct the predicted bolt tension curve. A vertical line to the curve at the pretension magnitude, if the bolt tension serves as the abscissa, comprises the initial portion of the predicted curve.

Deriving the equations for flange thickness by use of the free body in Fig. 5,

$$\left(\frac{P}{2} \right) d = M_w + M_b$$

$$P = \frac{2}{d} \left(M_w + M_b \right) \qquad (1)$$

For the elastic range,

$$M_w = M_b = k\ S\ \sigma$$

$$P = \frac{4\ M}{d} = \frac{b\ t^2}{6}\ k\ \frac{4}{d}\ \sigma = \frac{2}{3}\ \frac{k\ b\ t^2}{d}\ \sigma$$

$$t_{el} = \sqrt{\frac{3}{2}\ \frac{P\ d}{k\ b\ \sigma}} \tag{2}$$

At the upper limit of the elastic range,

$$M_b = k\ M_y$$

$$P_y = \frac{2}{3}\ \frac{k\ b\ t^2}{d}\ \sigma_y = 4\ \frac{k\ M_y}{d}$$

$$t_y = \sqrt{\frac{3}{2}\ \frac{P\ d}{k\ b\ \sigma_y}} \tag{3}$$

For ultimate load condition,

$$M_w = M_p = \frac{b\ t^2}{4}\ \sigma_y$$

$$M_b = k\ M_p$$

$$P_u = \frac{2\ M_p}{d}\ (\ 1 + k\ ) \tag{4a}$$

$$= \frac{b\ t^2}{2\ d}\ (\ 1 + k\ )\ \sigma_y \tag{4b}$$

$$t_u = \sqrt{\frac{2\ P_u\ d}{b\ (\ 1 + k\ )\ \sigma_y}} \tag{5}$$

For strain hardening condition,

$$k\ M_w = M_b = kM_{st}$$

$$P_{st} \quad \frac{2}{d}\ M_{st}\ (\ 1 + k\ ) \tag{6}$$

Next the expression for the prying force on the bolt will be developed.

$$P' e = M_b$$

$$P' = \frac{M_b}{e} \tag{7}$$

For the elastic region of flange behavior,

$$P' e = M_b = \frac{P d}{4}$$

$$P' = \frac{P d}{4 e} \tag{7a}$$

For the upper limit of the elastic range,

$$P'_y = \frac{P_y d}{4 e}$$

$$= \frac{k M_y}{e}$$

$$P'_y = \frac{2}{3} \frac{k M_p}{e} \tag{7b}$$

For ultimate load condition,

$$P'_u = \frac{k M_p}{e} = \frac{k}{e} \frac{d P_u}{2 (1 + k)} = \frac{k d P_u}{2 e (1 + k)} \tag{7c}$$

For strain hardening condition,

$$P'_{st} = \frac{k M}{e} = \frac{k}{e} \frac{P d}{2 (1 + k)} = 2 \frac{k d P_u}{e (1 + k)} \tag{7d}$$

The total bolt tensions can be determined as follows,

$$\sum T = 2\,P' + P \tag{8}$$

For the elastic range,

$$\sum T = P\left(\frac{d}{2\,e} + 1\right) \tag{8a}$$

For ultimate load condition,

$$\sum T_u = P_u\left(\frac{k\,d}{e(1 + k)} + 1\right) \tag{8b}$$

Because of strain hardening the ratio of the bolt line moment to the web moment at loads higher than P_u will remain at "k". The curve is then linear from P_u upward and is limited only by the yielding of the bolts. The curve in this region above P_u is constructed by choosing an arbitrary P greater than P_u, finding the corresponding bolt tension and connecting this point to the bolt tension at P_u.

$$\sum T_{st} = P_{st}\left(\frac{k\,d}{e(1 + k)} + 1\right) \tag{8c}$$

Moment Bracket

Moment capacity of fastener pattern.--Using the previous equations obtained from considering Tee sections in tension, general equations can be derived for the action of the vertical flange plate of a moment bracket (Fig. 22). In the case of two bolts per row the force at each level of fasteners contributing to moment capacity of the fastener group is:

$$F = 2\,T + 2\,P'$$

Summing moments about the center of the compression area,

$$P_{bu}\, x = (2T - 2P')\,(p + 2p + 3p + \ldots + (n-1)p) + \left(m - \frac{y}{2}\right) \sum_{0}^{n} (2T - 2P') \qquad (9)$$

where,

$$y = \frac{\sum_{0}^{n} (2T - 2P')}{\sigma_y W} \qquad (10)$$

The above are general expressions for the action of a moment resisting pattern with two fasteners per row.

Compression stiffener.--If a bottom stiffener to resist the compression is used, the stiffener must have a minimum thickness equal to y. If no such stiffener is used, the y distance will be much greater. The bolt rows falling within the distance y/2 can not be considered as contributing to moment capacity.

Shear in the web.--Enough web area is needed to resist in shear the compression force at the bottom of the bracket. This will often dictate the limit of any diagonal boundary from load point to lower web edge that may be desired.

If a stiffener extends from the load point to the bottom of the web the shear distribution will be that experienced in the web of a rolled section since the stiffener acts as a flange. The maximum shear stress for such a case is approximately 1.15 V/A_w where A_w is the area of the web across a horizontal section (Fig. 25). If no such vertical stiffener is used, then the shear distribution will be roughly parabolic and the maximum shear stress will be 1.5 V/A_w, or almost half again as great as the case with a vertical stiffener. The web can be accordingly proportioned using the yield shear stress equal to $1/\sqrt{3}$ of the tensile yield stress.

Investigation of a typical bracket.--A bracket was fabricated (Fig. 22) and tested. Using a one inch flange meant that the ultimate moment to be expected was 946 kip inches (Equation 9). This was equivalent to a load of 94.6 kips at an eccentricity of ten inches. The tensile force in each bolt at ultimate would have been 27.2 kips (Equation 8b).

The friction force resisting shear at ultimate is equal to the coefficient of friction times the force in the fasteners at that load. This was computed to be 55.6 kips. Therefore at ultimate load there should be a shearing force on each bolt of:

$$F_s = \frac{94.6 - 55.6}{8} = 4.9 \text{ kips.}$$

From the interaction curve the ultimate tensile load would then be 36 kips. A smaller bolt could have been chosen for the design.

A 3/4 inch web plate was used, but since P_u was 35 kips a 1/2 inch plate would have satisfied tension.

Since the compression force at the lower end of the flange at ultimate was

$$C = \sum_{0}^{n} (2\,T - 2\,P') \qquad (11)$$

$$= 140 \text{ kips}$$

and the load point stiffener did not extend to the lower edge of the web (Fig. 22),

$$\tau_w = \frac{3}{2}\frac{(140)}{(0.75)(11)}$$

$$= 25.4 \text{ ksi} > \frac{\sigma_y}{\sqrt{3}}$$

The web had suffered general yield prior to the ultimate load; thus it was underdesigned in shear. Since the depth of the web was not reduced at any point, the above was the critical section for shear in this case.

Comments on bracket design.--The flange should not be connected in any manner to the top load bearing plate. This is to allow as much freedom as possible for the flange to deform.

There is a question of whether or not it is wise to pretension the bolts in excess of that load assumed for design if the elliptical ultimate relationship curve (Fig. 23) is utilized.

Tee Stub Moment Connection

General.--The design of a Tee stub for the moment connection shown in Fig. 33 having a full tensile load of M_p /h follows the methods previously given involving the use of Equations (1) through (8).

To arrive at an expression for the friction force resisting shear, the tension stub has to be considered separately from the compression stub.

The flange contact force of the tension stub (Fig. 33) at ultimate is 2 P'_u; therefore,

$$Q_1 = 2 P'_u$$

The flange contact force of the compression stub is the summation of T forces plus the external force, or,

$$Q_2 = \sum T_i + P_u .$$

Therefore the total friction force resisting slip is

$$Q = \mu (2 P'_u + P_u + \sum T_i) \tag{12}$$

As strain hardening occurs with rotation, the force resisting slip increases.

The result of the bracket test indicate that not more than a few kips of tension per bolt will be lost by relaxation.

Design example.--The floor beams in a certain structural system are designed plastically for an ultimate load of 400 psf. This corresponds to a working load of 200 psf with a safety factor of two. The beam spacing is eight feet on centers. The beam required is an 18 WF 50 at a span of 29 feet. The plastic moment existing at the end is then 169 kip-feet. The corresponding shear at the connection is 47 kips. The design for the Tee stubs proceeds as follows.

Given that flange width of the Tee is nine inches, e will be 1.75 inches assuming a gage of 5 1/2 inches.

$$P_u = \frac{169\ (12)}{18.5} = 109.5 \text{ kips}$$

$$t_w = \frac{P_u}{\sigma_y\, b}$$

$$= \frac{109.5}{33\ (9)} = 0.37 \text{ inch}$$

Use 1/2 inch web

k is approximately $\dfrac{9 - 2(15/16)}{9} = 0.791$

From Equation (5),

$$t_u = \sqrt{\frac{2(109.5)2.5}{9(1.791)33}} = 1.02 \text{ inches}$$

Try a ST 18 WF 80 with flame cut edges.

From Equation (8b),

$$\sum T_u = 109.5 \left(\frac{0.791 \times 2.5}{1.75(1.791)} + 1 \right) = 179 \text{ kips}$$

Therefore try four one inch diameter high tensile strength bolts.

Check:

$$k = \frac{9 - 2(1.0625)}{9} = 0.761$$

$$t_u = \sqrt{\frac{2(109.5)2.43}{9(1.761)33}} = 1.01 \text{ inches}$$

Use the ST 18 WF 80.

$$\sum T_u = 109.5 \left(\frac{0.761 \times 2.43}{1.75(1.761)} + 1 \right) = 175 \text{ kips}$$

$$T_i = T_u = \frac{175}{4} = 43.8 \text{ kips}$$

Use four one inch high tensile strength bolts and pretension to 49 kips as recommended(1).

The force resisting slip as computed by Equation (12) using a coefficient of friction of 0.25 is 93 kips. This is conservative and yet is almost twice as much as the shear encountered in this typical situation.

CHAPTER III

DESCRIPTION OF TESTS

Tee Sections

Assembly.--The general test set-up is shown in Fig. 1. The relatively inflexible base Tee that served for all the flexible flange specimens consisted of a Tee section of similar dimensions to the flexible Tees except that it had a flange thickness of two inches. Both the base Tee and the test Tee were butt welded to tension grips, and the changing of flexible Tees involved flame cutting the tested specimen from the grip and welding on a new specimen.

The base Tee and the specimen Tee were connected by the particular size high strength bolts used in the test through 1/16 inch oversize match drilled holes. In test T-5, where 5/8 inch diameter high tensile strength bolts were used, bushings were placed in the base Tee holes, reducing their diameter from 15/16 inch to 11/16 inch. One hardened washer was placed under each bolt head and nut.

A 450,000 lb capacity Riehle Mechanical Testing Machine was used to provide the test load. A strain rate of 0.025 inches per minute was used.

Bolts.--The bolts used were manufactured in accordance with ASTM designation A325 (Ref. 3). The bolt dimensions conformed to the current requirements for regular semifinished Hexagon Bolt of the American Standards Association (ASA designation: B18.2) (Ref. 5). The 7/8 inch bolts had a grip of 3 3/16 inches and a length of 4 1/2 inches. The 5/8 inch bolts had a grip of 3 3/16 inches and a length of 4 1/2 inches.

Washers.--The washers used were circular, flat, and hardened. The dimensions conformed to the current requirements of the American Standards Association (ASA designation: B27.2) (Ref. 1).

Nuts.--The nut dimensions conformed to current requirements for Heavy Semifinished Hexagon Nuts of the American Standards Association (ASA designation: B18.2) (Ref. 5).

Specimen Tees.--The structural Tees used were some that had been used for a previous master's thesis on the behavior of a moment carrying beam connection. These Tees were machine flame cut from an 18 WF 85 shape, and as seen from Table 2, the web was rolled off center to the limit of rolling tolerance(6).

The flange thickness of specimens T-1 and T-6 was left as rolled, while that of T-2, T-3, T-4, and T-5 was machined to the thickness as indicated in Table 2.

The flange width was left untouched except for specimen T-4 for which the width was reduced in order to study the effect of variation in edge distance.

The gage distance was 5 1/2 inches in all cases. Edge distances are shown in Table 2. Distance between holes in line was also 5 1/2 inches except for T-5, which used six 5/8 inch diameter bolts. In this case the hole distance was 2 3/4 inches. In all cases the holes were placed symmetrically with respect to the length of the Tee.

Assembly.--A welding jig was built so that the base Tee, specimen Tee, and the tension grips could be pre-assembled in line to insure an axially loaded test.

Assembling of the first test assembly involved bolting the base Tee and specimen Tee together, and welding the grips on each in line.

The following tests involved only bolting the specimen Tee to the base assembly and welding in line the specimen tension grip.

The final assembly consisted of two sub-assemblies: a base Tee with grip, and a specimen Tee with grip. After the welds had cooled sufficiently the bolts were removed in order to place the assembly in the testing machine in two component parts. The base Tee was suspended from the fixed head of the testing machine and the specimen Tee placed in the moveable head. When the flanges had been brought together the bolts were placed and torqued to the desired pretension load. The bolt calibration curves are shown in Figs. 6 and 7. The bolts were torqued by hand until the desired elongation occurred.

Instrumentation.--A two inch Whittemore Gage was used to measure the flange separation and compression edge action of the flange (Fig. 2).

Six-inch micrometer calipers with ratchet and pointed tips were used to measure elongation of the bolts. Each end of the bolt was center drilled to provide a machined seat for the caliper tips. By this arrangement it was possible to repeat readings to within 0.0001 of an inch. This was considered sufficient accuracy since the bolt alongation at the elastic proof load was about 0.0080 of an inch.

A temperature compensating reference bolt was clamped to the assembly for tests T-3, T-4, T-5, and T-6 after it was observed that a variation in the temperature of the calipers caused by handling affected the readings somewhat during the tests of T-1 and T-2.

Several coupons were cut from the approximate positions of the flange as shown by Fig. 3 and their strengths tabulated in Table 1. The results are plotted in Fig. 3 and indicate the variation in yield strength that occurred across the flange. These results were checked by several coupons.

Tee test procedure.--The procedure began by taking a zero reading of the length of the bolts prior to torquing. After torquing of the bolts, and at zero load on the testing machine, readings were taken on the bolts with the calipers and on the flange with the Whittemore Gage at positions indicated in Fig. 2. The loading proceeded at approximately fifteen increments between zero load and the expected flange yield load. At each increment readings were taken as outlined above.

Abrupt failure due to stripping of the threads of a nut occurred in tests T-1 and T-2. In the succeeding tests it was decided to load until the flange had yielded but to stop loading when the bolt elongation showed forty-five or fifty kips of tensile force in each bolt. This was possible because calibration of these particular bolts showed that the actual elastic proof load was at least 18 kips above specified minimum proof load. Load on the test assembly was reduced to zero, readings were taken again to observe permanent deformation, and the bolts were taken out to be used again in succeeding tests. On none of the tests did the 7/8 inch bolts exhibit any excessive permanent set. The 5/8 inch bolts of test T-5, however, could not be loaded far beyond their minimum elastic proof load without suffering excessive permanent set.

Test T-1.--The primary purpose of this test was to see how close the yield load could be predetermined by using an investigation based on simple plastic theory. In this test the flange was left as rolled. The dimensions used in the computations are averages of symmetrical dimensions on the actual Tee.

From Fig. 3 it can be seen that the yield stress is approximately 34 ksi in the regions that the plastic hinges could be expected to form, that is, at the bolt line and the edge of the web.

$$P_u = \frac{b}{2}\frac{t^2}{d}(1 + k)\,\sigma_y \tag{4b}$$

$$= \frac{8.875}{2}\frac{(0.908)^2}{(2.47)}(1.8)34$$

$$= 90.6 \text{ kips}$$

The load-deflection curve (Fig. 8) shows that excessive deformation started near this load. This curve also shows good agreement with the predicted value of first yield.

No curve is presented for load vs. bolt tension because of the temperature effect already described in the section on instrumentation. The initial tension was not far from the proof load, however, and the tension in the bolts began to increase between a load of 85 and 90 kips.

Test T-2.--Dimensions of T-2 are shown in Table 2. The flange of this specimen was machined to an average thickness of 0.798 inch. Using Equation (4b) the failure load is found to be 69.7 kips. This load is defined as that which causes excessive deformation of the flange. Fig. 9 shows the computed failure load indicated on the load-deformation curve.

The test conditions were similar to those of T-1, and again it was impossible to know the exact pretension placed on the bolts. In both cases, however, this initial tension was not far from the minimum elastic proof load for 7/8 inch high tensile strength bolts. The minimum elastic proof load is 37 kips(1).

Test T-3.--The flange in this case was machined to an average thickness of 0.851 inch. In addition, a low initial tension of approximately 0.4 of the elastic proof load was placed on the bolts. This was done to study the correlation between predicted and actual bolt tension curves at low loads. In this and succeeding tests the exact tension in the bolts was known at all stages of loading.

The failure load, computed to be 78.4 kips from Equation (4b) is shown on the load-deflection curve Fig. 10.

Fig. 14 shows a comparison between the average tensile load per bolt computed from Equation (8) and the actual average tensile load. At predicted ultimate load the bolt tensions were 11% lower than predicted.

Test T-4.--In order to study the effect of a varying edge distance on the validity of analysis by Equations (4b) and (8) the edge distance was narrowed to 1 1/8 inches. This is the minimum allowed by AISC specifications(6).

The average thickness used in computing the failure load of the flange was 0.831 inch. The four bolts were torqued to an average initial tension of 39.6 kips. This was 22 per cent in excess of the minimum required bolt tension of 32.4 kips as recommended by specifications(1). These specifications recommend a pretension load approximately 15 per cent in excess of this required minimum load in order to assure a minimum tension equal to the elastic proof load.

Fig. 11 shows the degree of accuracy by which Equation (4b) predicts the yield load of the flange. Fig. 15 compares the theoretical and actual bolt tension curves.

Test T-5.--In this particular test specimen six 5/8 inch diameter high tensile bolts were used instead of the four 7/8 inch bolts used in the other four tests. These bolts were pretensioned to an average of 20.1 kips.

Preliminary calculations indicated that a flange failure load of 76.6 kips was to be expected, and at this load there should be approximately 20.5 kips of tension in each of the six bolts. The results are shown in Figs. 12 and 16. Fig. 7 is the calibration curve for the 5/8 inch bolts used in this test. These bolts exhibited a more nearly normal yield level than the 7/8 inch bolts used in the other tests. The effect of this yielding is evident in Fig. 16.

Test T-6.--T-6 was an attempt to duplicate the conditions of T-1. The flange thickness was left as rolled. Flange failure load was computed to be 89.6 kips and the average tension per bolt at that load computed at 36.3 kips. Figs. 13 and 17 show the results.

Bracket

General.--The bracket designed in the chapter on theoretical development was fabricated of plates connected by welds (Fig. 22). It was mounted with 7/8 inch diameter high tensile strength bolts to a two inch plate which was welded to the end of a 24 WF section. The testing arrangement is shown in Fig. 21. The bracket was mounted in an inverted position so that it could be loaded by one of the reactions to a concentrated load applied to the rolled section at a point six inches behind the two inch mounting plate. The contact surfaces were thoroughly cleaned of mill scale prior to mounting.

The load was applied by a 450,000 lb capacity Riehle Mechanical Testing Machine at a strain rate of 0.025 inches per minute.

The bolts were initially tensioned to approximately 0.9 of the elastic proof load in accordance with the assumptions made in the design of the bracket.

Instrumentation.--The eight 7/8 inch bolts used were center drilled as in the Tee tests. Micrometer calipers were used to measure elongation.

Micrometer mechanical dials were used to measure the rotation of the bracket at two places: (1) the centerline of the flange with respect to the mounting plate, and (2) the rear extremity of the bracket with respect to the mounting plate (Figs. 20 and 21).

The photograph of Fig. 19 was taken before the flange rotation dials were mounted.

Two Whittemore Gage points were located to study the compression edge action at lateral points on the bracket flange (Fig. 21).

Dials were mounted on the bracket on both sides to measure the slip. The holes were misaligned so that the maximum possible slip could occur. A dial also measured the deflection of the 24 WF section at a point underneath the primary load point.

A reference standard bolt was used for temperature compensation.

Procedure.--Loading progressed at approximately twelve-kip increments on the bracket until the first slip occurred at 96.5 kips. This slip was accompanied by a very sharp report. The bracket load immediately fell off to 56.5 kips at which time instrument readings were taken and loading was resumed. At 158 kips, about 67 per cent above predicted ultimate, the loading was stopped. This load had dropped to 153 kips by the time data was collected. At this point the load was allowed to remain on the bracket for thirteen hours before resuming the test. This interruption of loading was used to investigate any change of conditions that might have occurred over this interval. After the delay in testing the bracket load was taken back to 158 kips as the first increment and showed less than 0.5 kip drop-off by the time data was collected.

Loading then progressed to 172 kips whereupon another major slip occurred. This was also accompanied by a sharp report. The load fell off to 144 kips at which time readings were taken and loading was again resumed. At 175.5 kips the butt weld connecting the web with the flange failed in shear. Inspection of the fractured weld showed it to be defective due to lack of penetration into the grooves provided for the butt weld.

CHAPTER IV

DISCUSSION OF RESULTS

Tee Sections

Deflection curves.--In all instances where the breakpoint on the deflection curve was sharply defined it was in good agreement with the predicted value as given by Equation (4b). The curve of test T-3, which had the sharpest break at ultimate, shows how close simple plastic theory concurred with test results.

Although Whittemore Gage points were located so as to study the effect of compression edge action, the resulting curve, though smooth, shows no results significant enough to be presented. There is an indication, however, of a definite compressive yielding at the point of contact.

Fig. 18 exhibits the coincidence of the theoretical deflection of a point on the flange at loads below that which produced first yield at a hinge position. At first yield the curves diverge. The elastic deflection was computed on the basis of a fixed end beam with a span equal to the gage dimension minus web thickness.

If a safety factor of two were used against the ultimate load occurring, the deflection in this case would still be in the elastic range.

Bolt tension.--It seems evident that Equation (8) is able to predict with sufficient accuracy the tension in the bolts at any load. There is evidence from Test T-6, however, that an initial overstress will produce slightly harmful effects, in that the curve of actual tension breaks from the pretension load at ultimate instead of continuing upward to the predicted curve (Fig. 17) as might be expected under the assumptions of these tests. Since the actual curve after break-off follows practically the same slope as the predicted curve, it is possible that this one test is not indicative of such a conclusion. At most there is approximately fifteen per cent difference between a computed tension and actual tension.

It is notable that the collapse mechanism allows analysis at loads greater than ultimate, which can occur because of strain hardening. This is predicated on the assumption that the moments on the free bodies of Fig. 5 will remain at the ratio of "k" at higher loads. The assumption is not exactly true because of another transition zone that occurs as one hinge begins to strain harden while the other is yielding freely. This zone is probably insignificant, since strain hardening occurs at the outer fibers of a hinge almost immediately as it begins to rotate.

The importance of the transition zones diminishes, since inspection of a final predicted tension curve reveals it to be practically a straight line.

Bracket

Bolt tension.--At the computed ultimate load the bolts did not increase in tension excessively except for bolts seven and eight which gained about four kips. A report by Fuller and Munse(4) shows that no deleterious effects were experienced by overtorquing 7/8 inch diameter high tensile strength bolts to as much as 56.5 kips per bolt. Their particular bolts were tested statically and in fatigue and had an elastic proof load of 37 kips.

The two rows of bolts nearest the compression edge dropped slightly in load due to relaxation. It was noticed prior to mounting that the flange warped a slight amount with the concave side towards the mounting plate. Torquing the bolts then apparently did not completely correct the condition.

When slip occurred the two rows that were exhibiting a tendency to decrease immediately gained load and the two rows that were gaining tension lost load. Therefore, slip effected a redistribution.

Bolts five, six, seven, and eight showed a slight tendency to increase in tension prior to computed ultimate (Figs. 28 and 29). Coupons cut from the excess material that had been used for the flange showed that the yield strain of the stress-strain diagram was only four times the elastic range compared with the ratio of 15:1 expected of steel that is to perform according to plastic concepts. This lack of a flat yield zone meant an early strain hardening of the steel as it was subjected to rotation in the vicinity of hinge formation.

At the secondary slip another redistribution of load occurred.

Conditions were stagnated during the period that loading was suspended for thirteen hours. This was done at a load 67 per cent in excess of predicted ultimate. Except for loading the bolts, the effect of strain hardening is beneficial.

Slip.--The first slip occurred at 96.5 kips, or slightly over the computed ultimate load (Fig. 30). The contact surfaces were sanded free of mill scale. The bearing force on the flange at ultimate equalled the total amount of tension in the bolts or 245 kips. The apparent coefficient of friction then was 0.39. This was probably due to the misalignment of holes and a "digging in" that takes place at the compression edge of the flange.

Slip would not have occurred at the working load in this case.

It is likely that the major slips occurred in two phases: slip of the bracket with respect to the bolts, and slip of bracket and bolts with respect to the base plate.

Web shear.--The assumption of maximum shear intensity in the web at the compression area of the flange is remarkably close to test results.

If the load point stiffener does not extend to the bottom of the web so that it cannot transmit shear flow, as in this case, then at shear yield from the equation that shear stress equals 3/2 V A,

$$C = V = \frac{2}{3} \tau_w A$$

$$= \frac{2}{3} \frac{(33)}{\sqrt{3}} (0.75)(11)$$

$$= 105 \text{ kips}$$

The bracket load at shear yield should be, from Equation (11), 105/140(94.6) or 71 kips. The first yield lines in the whitewash were observed at a bracket load at 76 kips and assumed roughly a parabolic distribution. The shear conditions shown by Fig. 20 had occurred at 158 kips at which time strain hardening in shear must have already occurred.

Rotation.--The rotation curves (Figs. 31 and 32) show that the ultimate load as computed by Equation (9) agrees with test results.

At the possible working load rotation was hardly discernable.

The shear deformation in the bottom of the web gave a false impression of the magnitude of the load-point rotation. The load-point rotation dials were mounted on opposite sides of the zone which yielded excessively in shear. The results, however, show that such shear failures are not catastrophic.

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

1. The method of analysis by the application of simple plastic theory is adequate for describing the flexing action of a connection flange and resulting tension in the fasteners.

2. The effect of strain hardening of the flange on fastener tension can be computed by the equations developed.

3. It is possible to predict the bolt tension at any load in a flexible flange connection with sufficient accuracy for design purposes. The tension in a fastener depends on the strength characteristics and geometry of the flange and statics in such a manner that application of plastic theory can be used for its determination.

4. Initial pretension of fasteners has no effect on the ultimate load of high tensile strength bolted flange type connections. The fastener tension will follow a predetermined curve as soon as the initial tension is overcome.

5. The methods presented are effective in designing a moment bracket based on ultimate concepts and computing the ultimate load with accuracy.

6. The apparent coefficient of friction is high in a situation such as that of a moment bracket. Misalignment of holes and a "digging in" at the compressive yield zone are major contributing factors. For the moment bracket tested the apparent friction coefficient was 0.39.

A high tensile strength bolted connection can be depended on to carry the load by friction throughout the ordinary working load range.

7. It is possible to apply these ultimate concepts in the design of a Tee stub moment connection. A typical design is presented in the chapter on theoretical development.

8. It is recommended that designs incorporating the results of this investigation consider the following points:

a. Equations (4) and (5) can be used to correlate flange thickness and ultimate load.

b. Choosing an available flange thickness slightly smaller than computed will not endanger the design.

c. A fastener size should be chosen sufficient to resist the tension through the fastener hole at ultimate as given by Equation (8b).

d. The friction force from initial bolt tension can be depended on to resist slip. This force is conservative, using the normal coefficients of friction for the surfaces in contact.

e. The edge distance should be kept large to reduce the prying force contributing to bolt tension.

A P P E N D I X

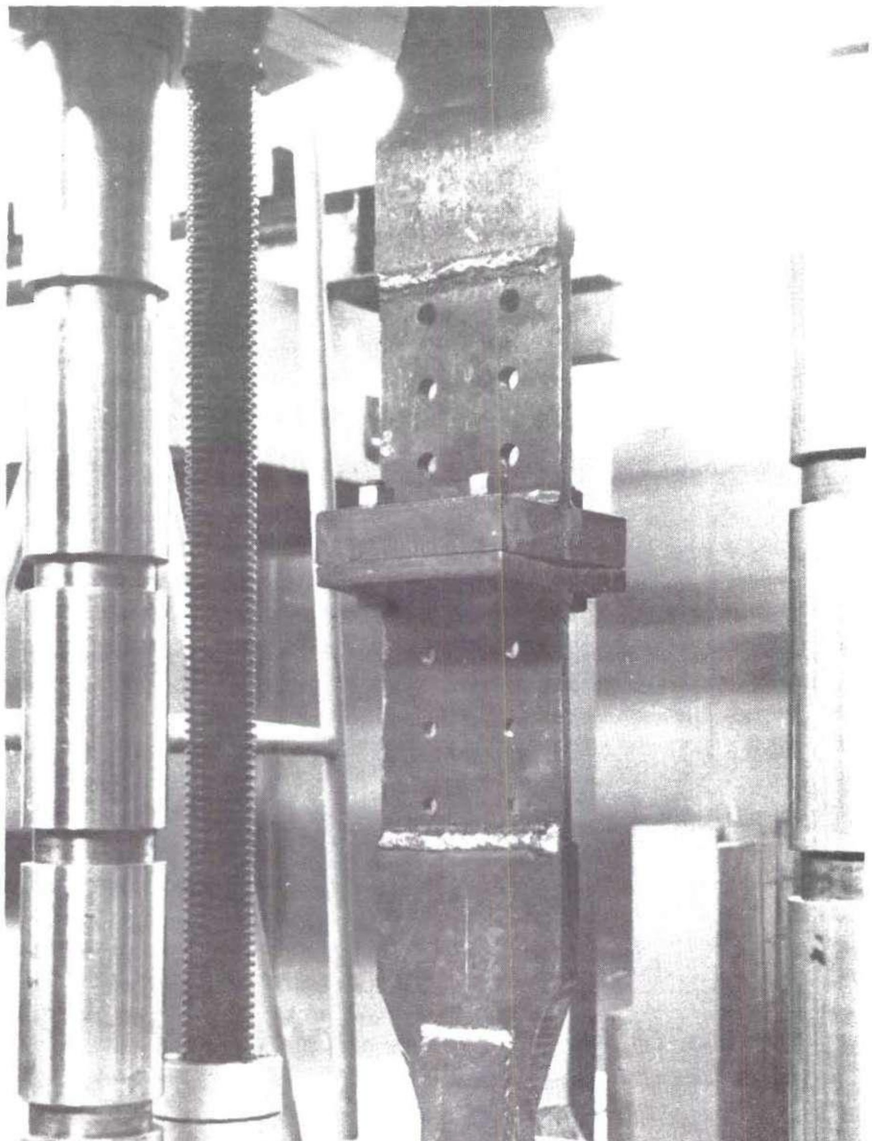


Fig. 1. Test Arrangement for Tee Sections

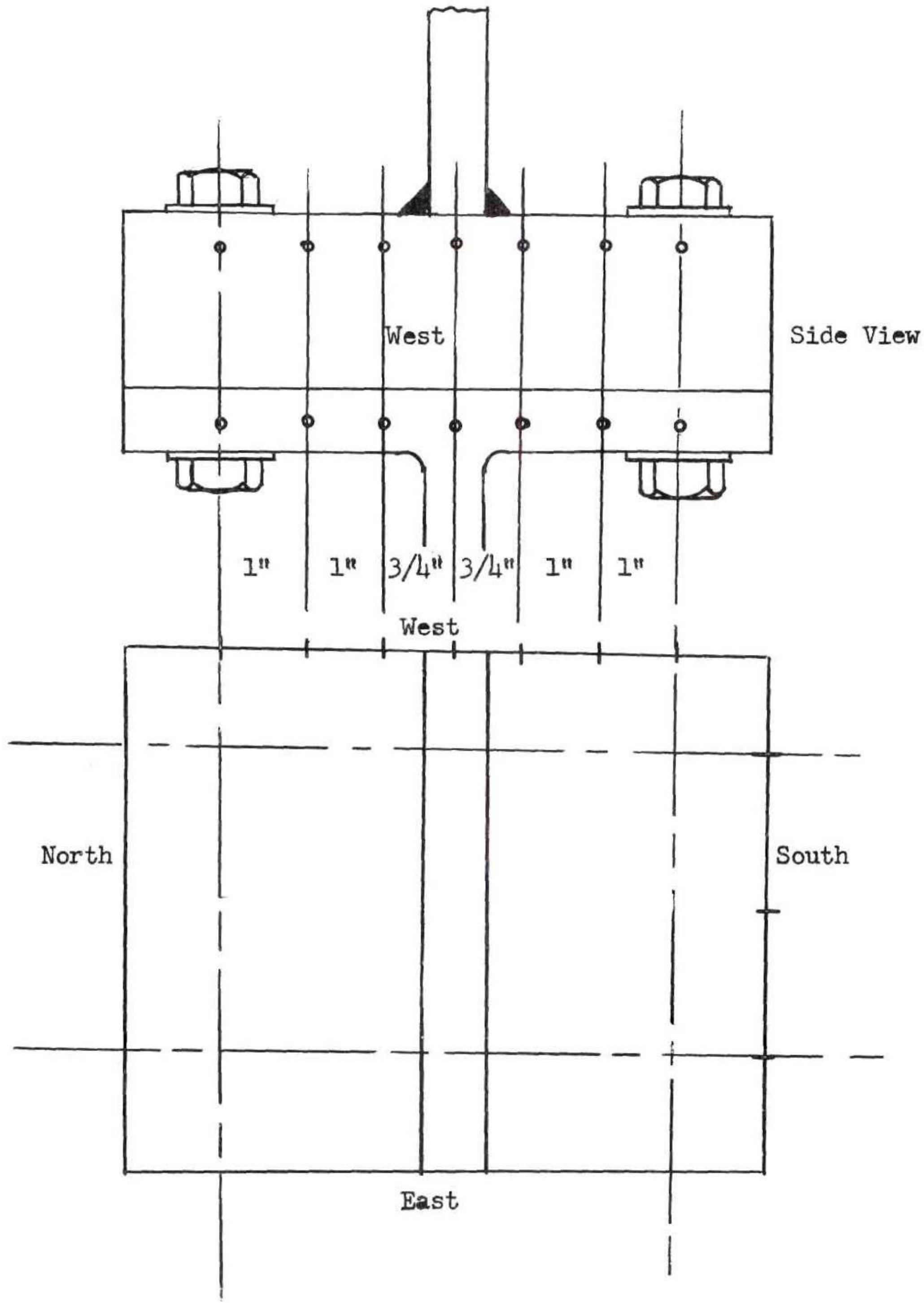


Fig. 2. Location of Whittemore Gage Holes for Tee Specimens

Table 1. Coupon Strengths

Coupon	Yield Strength in psi	Ultimate Strength in psi
E-1	36,400	Unknown
E-2	36,800	62,400
H-1	32,500	63,800
M-1	32,500	64,300
M-2	31,000	65,700
C-1	34,200	68,800
C-2	35,900	66,500
Bracket-1	31,800	62,600
Bracket-2	32,400	62,800

Table 2. Dimensions of Tee Specimens(Refer to Figure 4)

All dimensions are in inches

Item	A	B	C	D	E	F	G	H	J
T-1	4.00	4.44	0.913	0.900	0.909	0.909	8.875	1.750	1.750
T-2	4.00	4.44	0.815	0.793	0.793	0.788	8.905	1.750	1.750
T-3	4.00	4.44	0.845	0.838	0.864	0.858	8.813	1.750	1.750
T-4	3.66	3.63	0.829	0.800	0.845	0.849	8.813	1.125	1.125
T-5	4.18	4.32	0.866	0.860	0.823	0.847	8.750	1.750	1.750
T-6	4.03	4.41	0.907	0.907	0.912	0.908	8.813	1.750	1.750

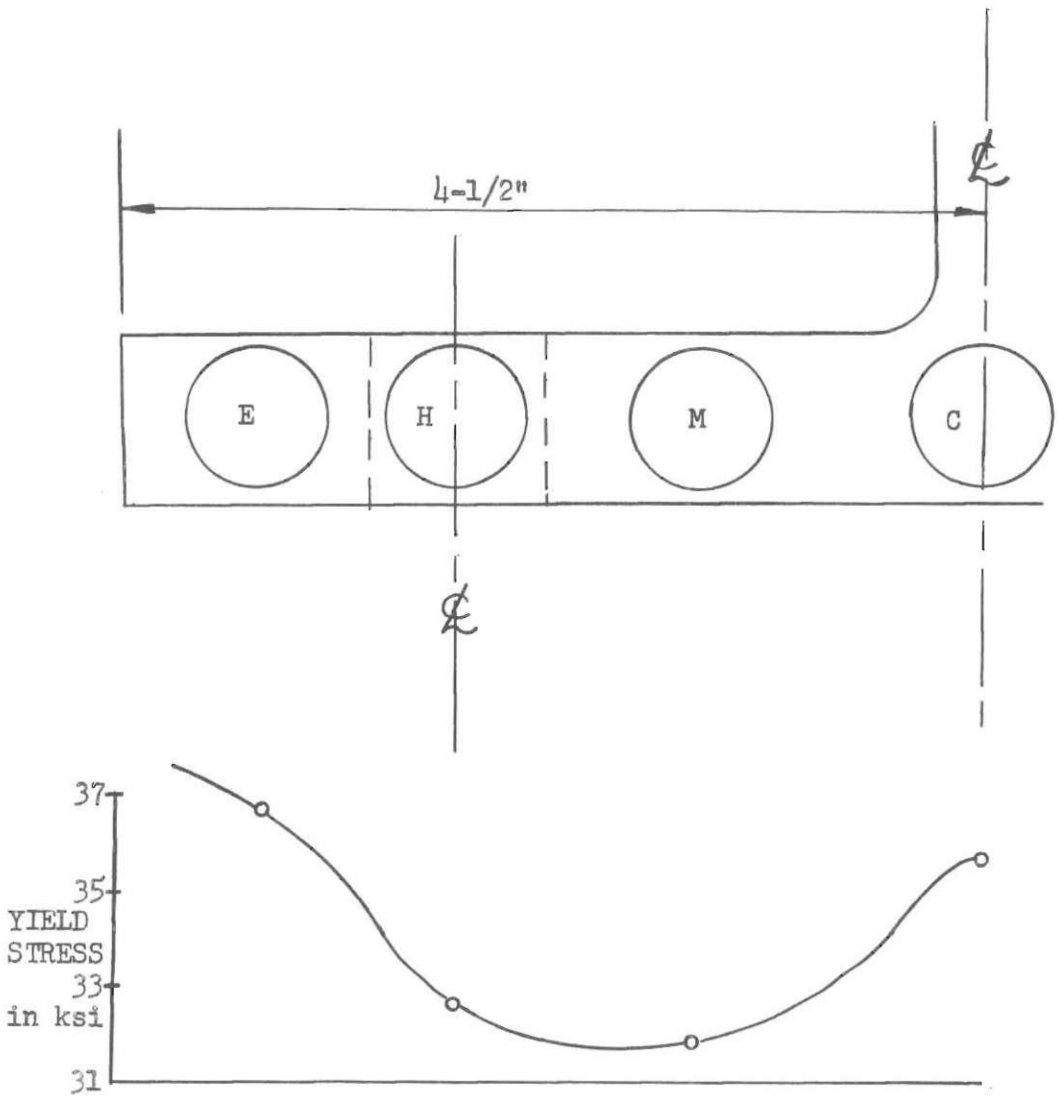


Fig. 3. Average Yield Stress Variation Across Flange of Tee Specimens

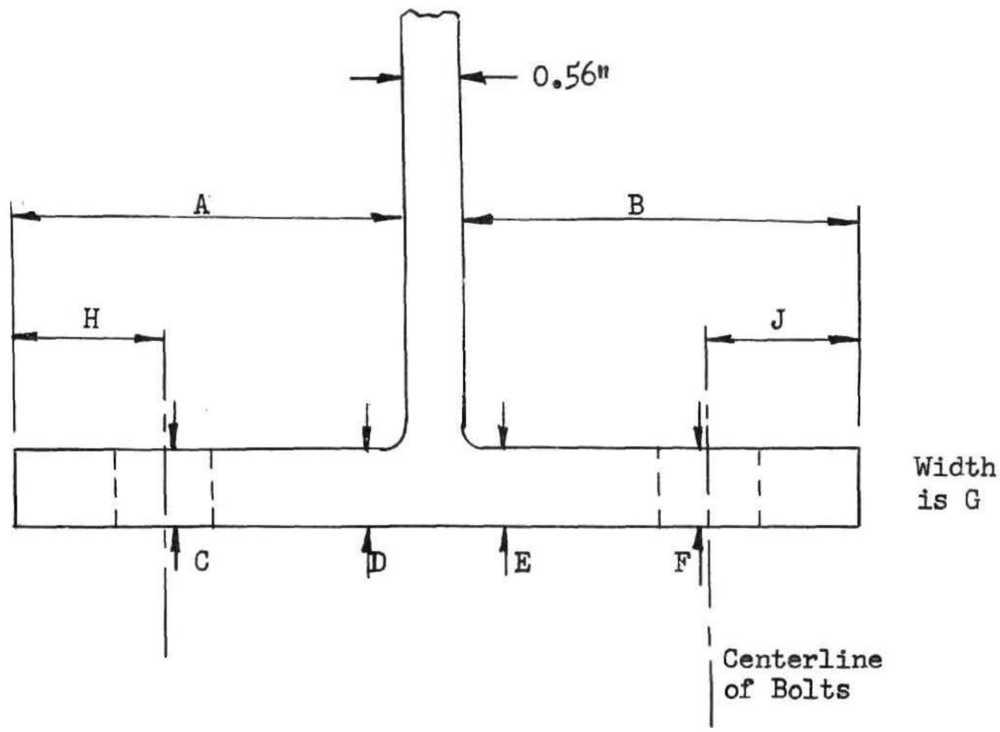


Fig. 4. Dimensions of Tee Specimens

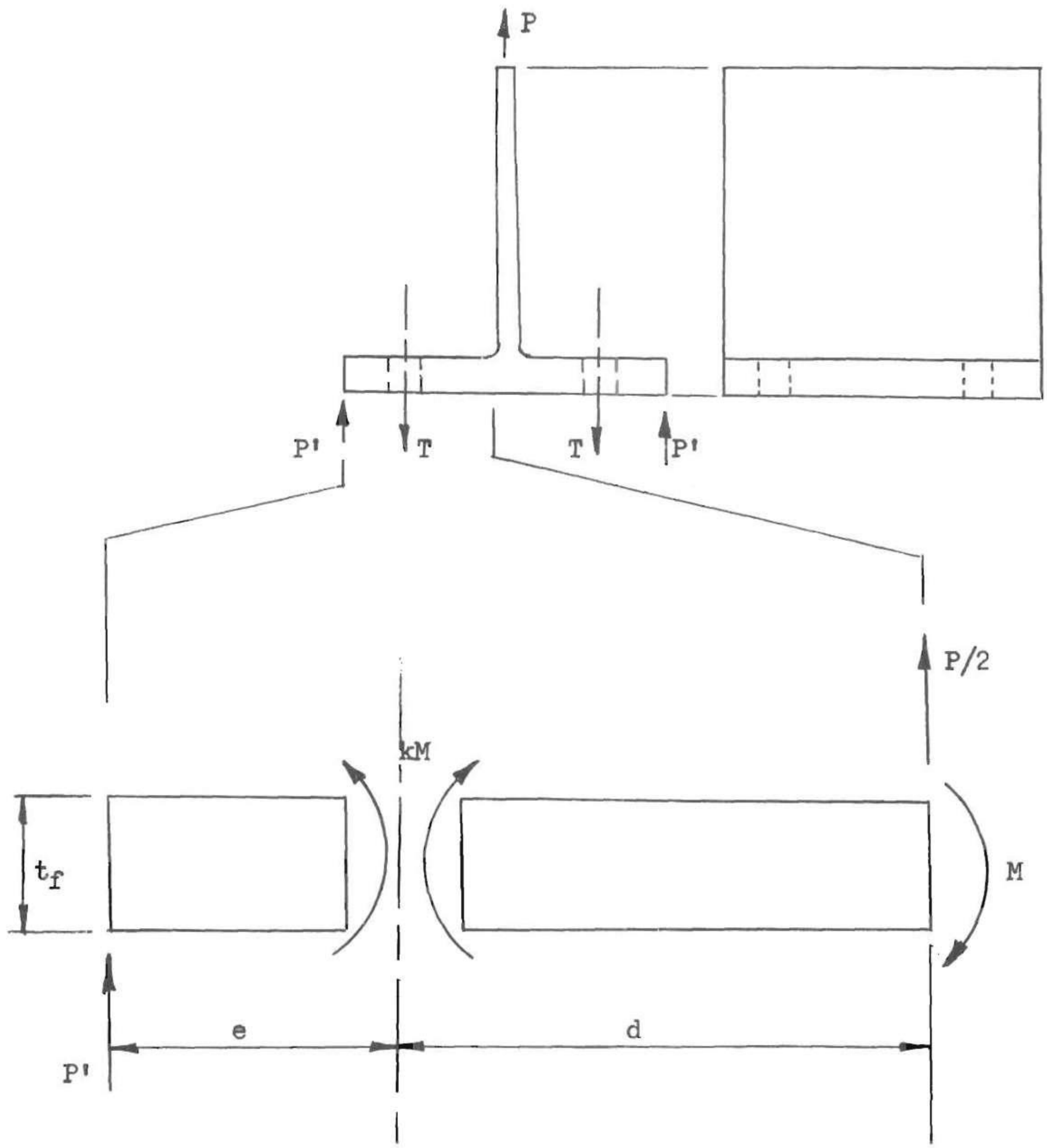


Fig. 5. System of Forces Acting on Tee Flange

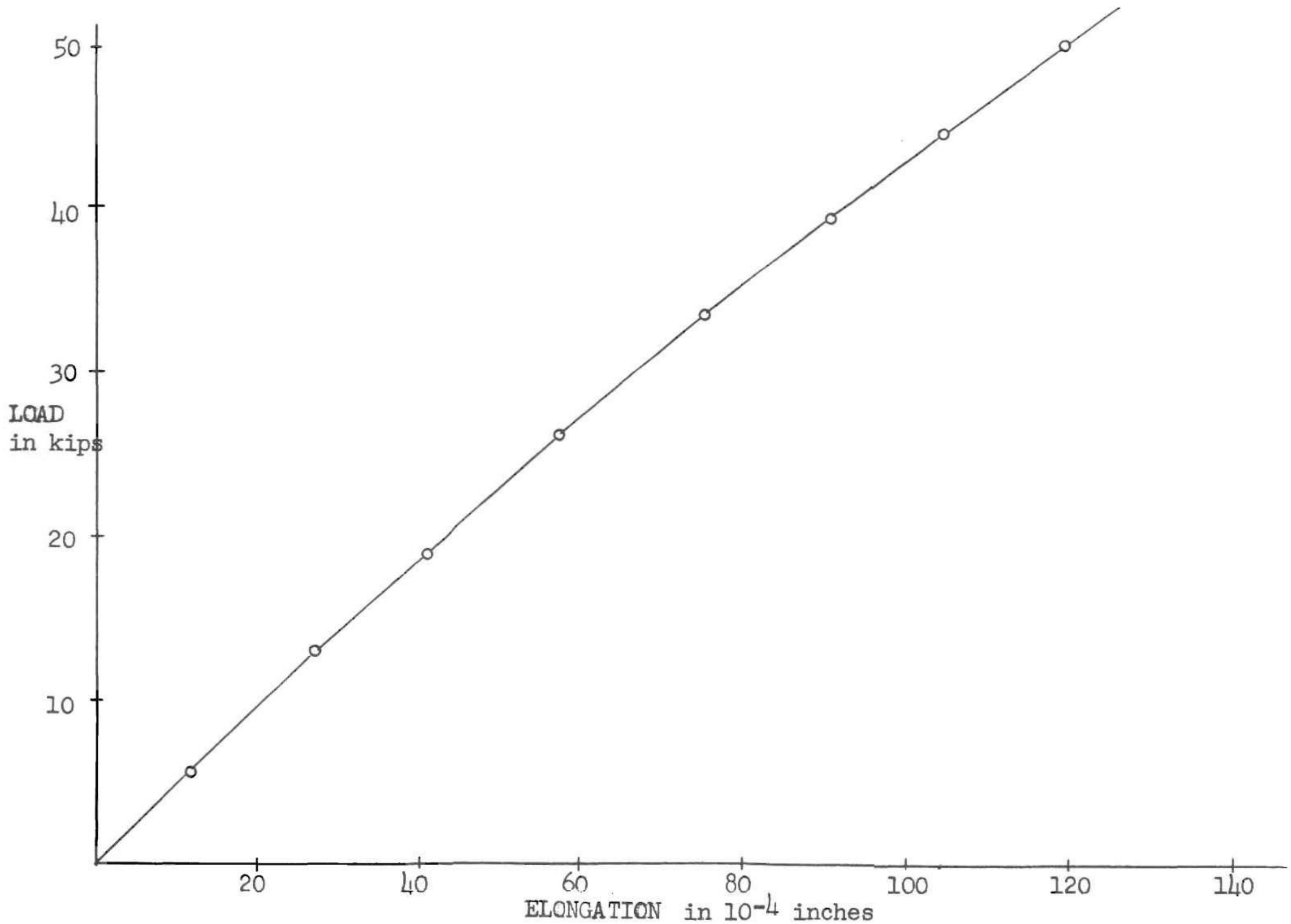


Fig. 6. Average Calibration Curve of the 7/8 inch Bolts Used

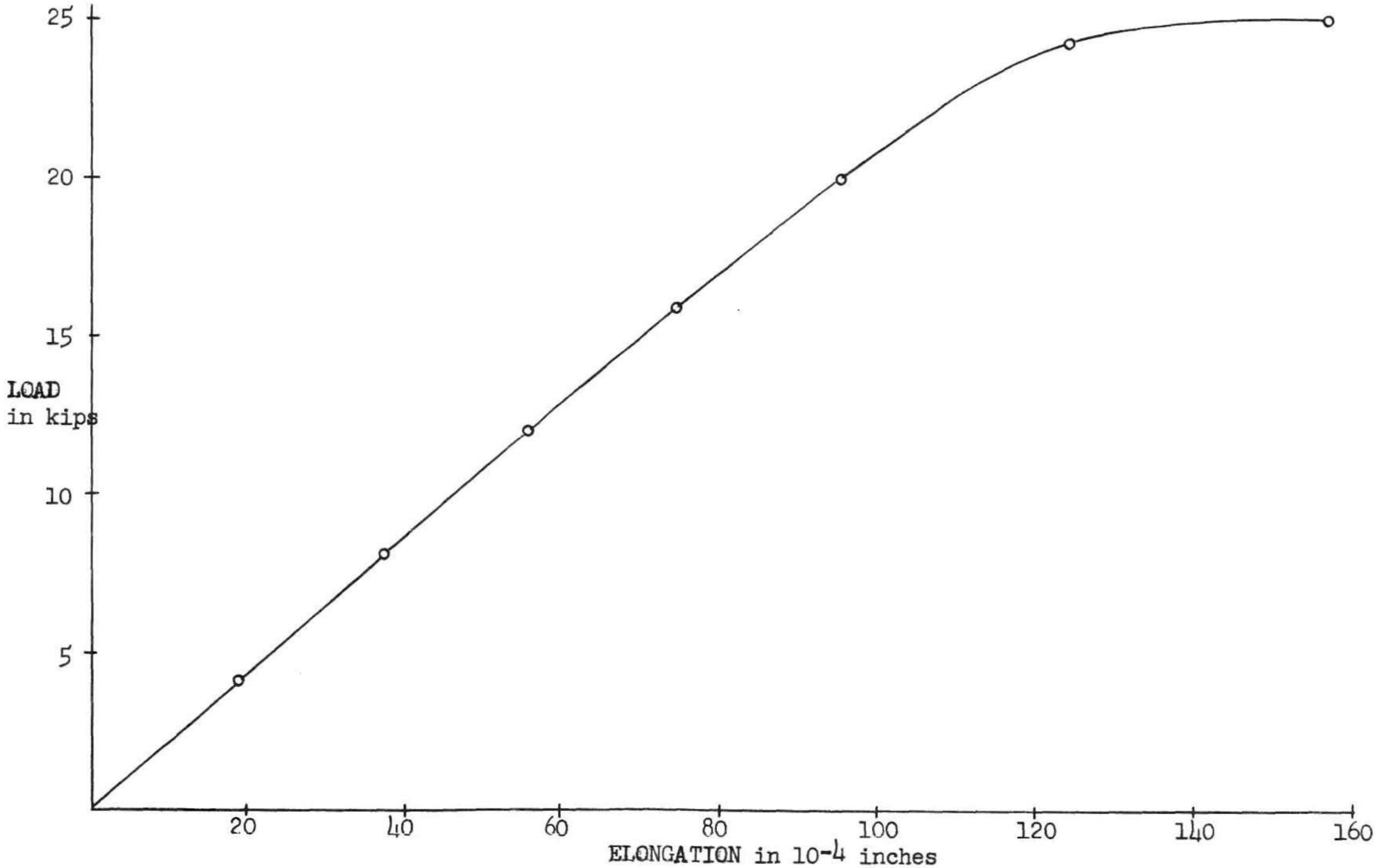


Fig. 7. Average Calibration Curve of the 5/8 inch Bolts Used

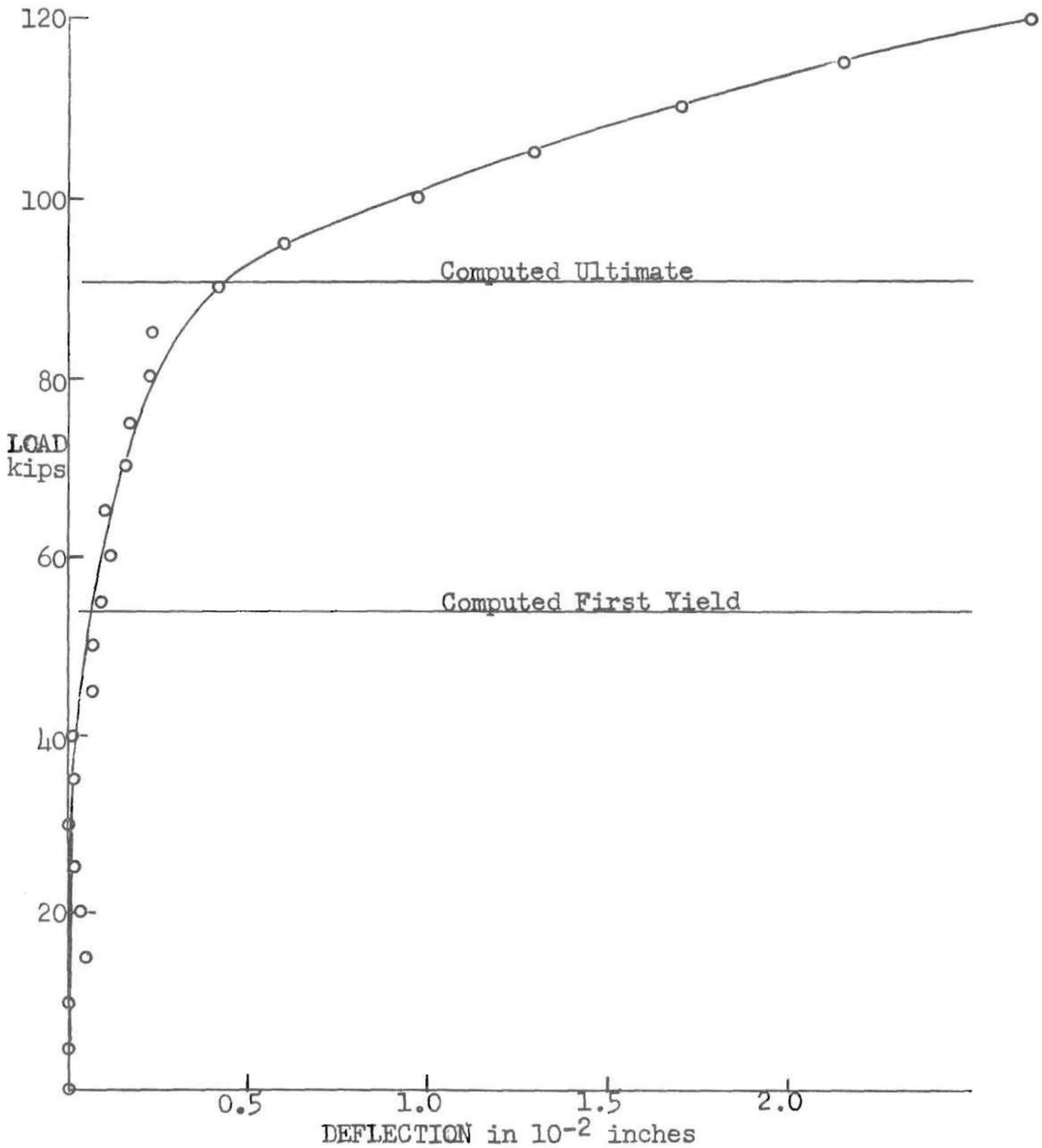


Fig. 8. Deflection of Flange Near Web- Test T-1

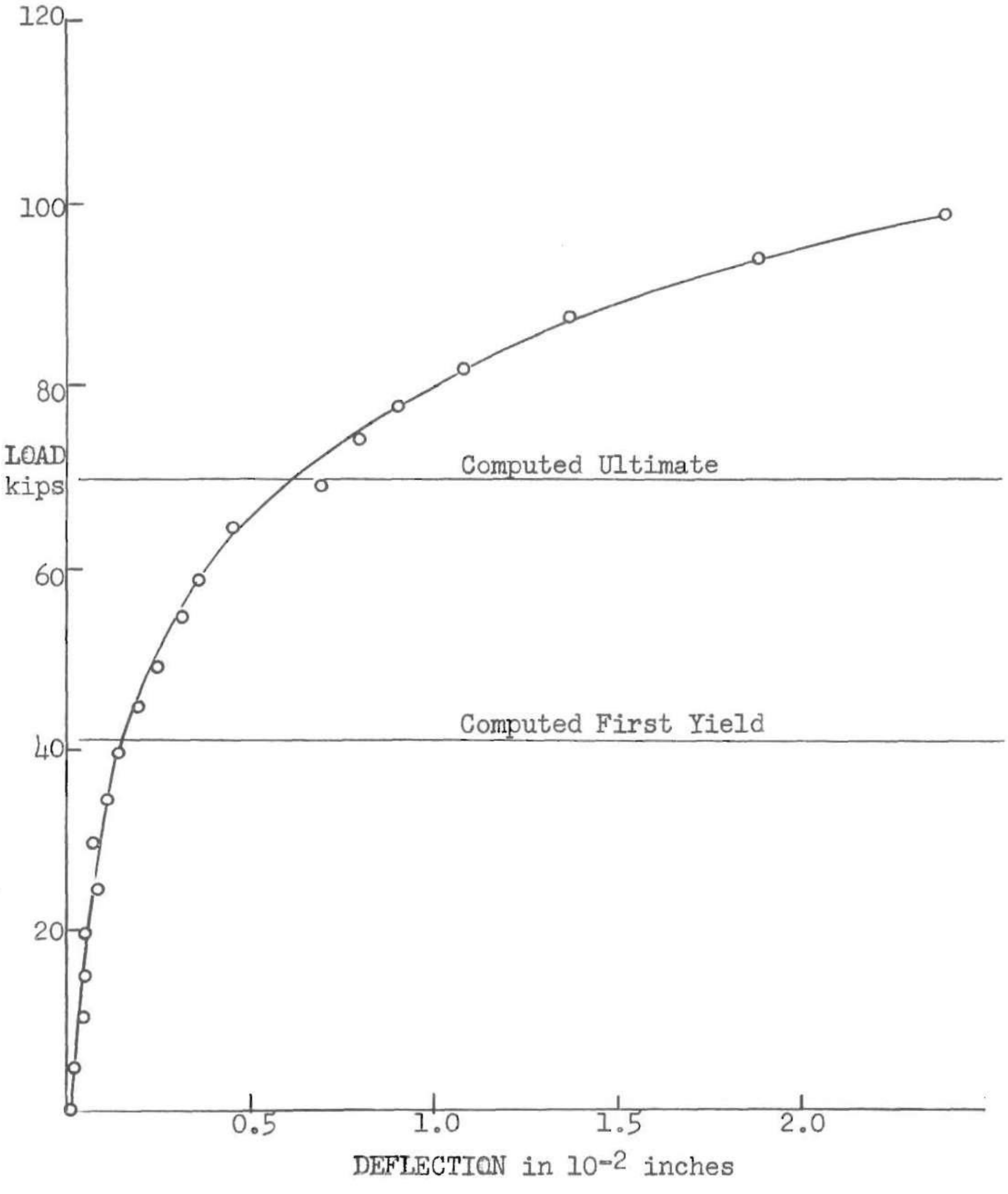


Fig. 9. Deflection of Flange at Web - Test T-2

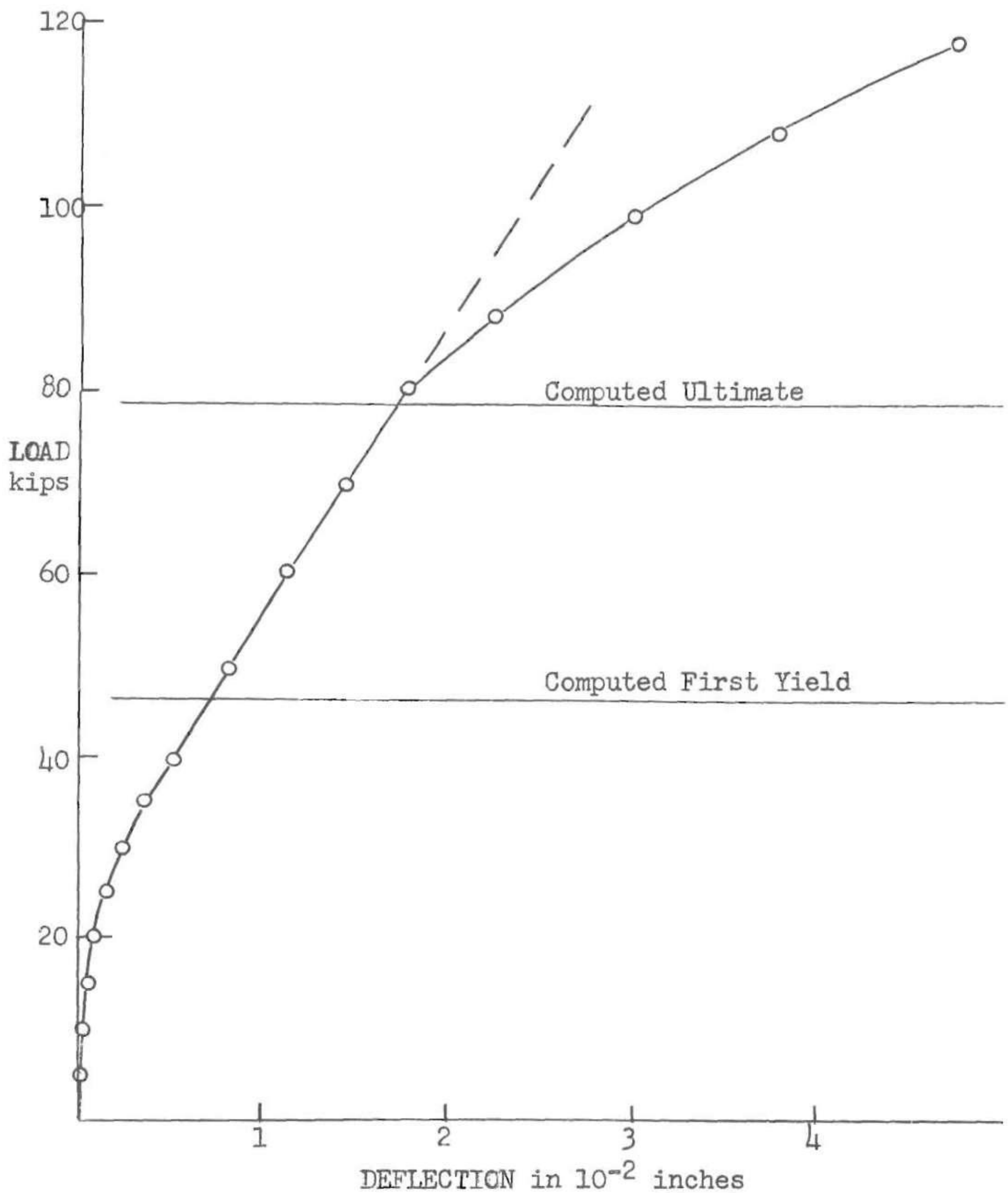


Fig. 10. Deflection of Flange at Web - Test T-3

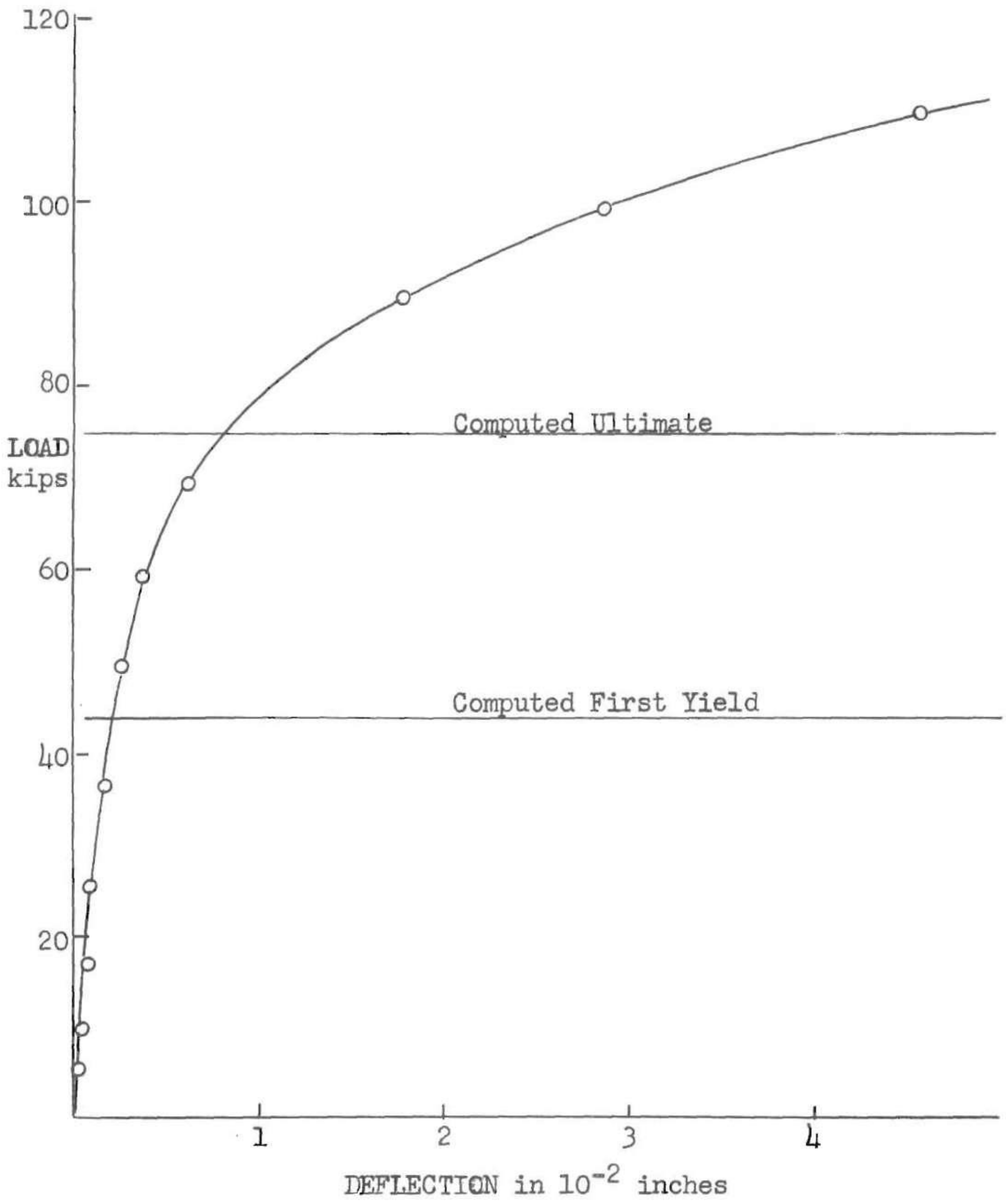


Fig. 11. Deflection of Flange at Web - Test T-4

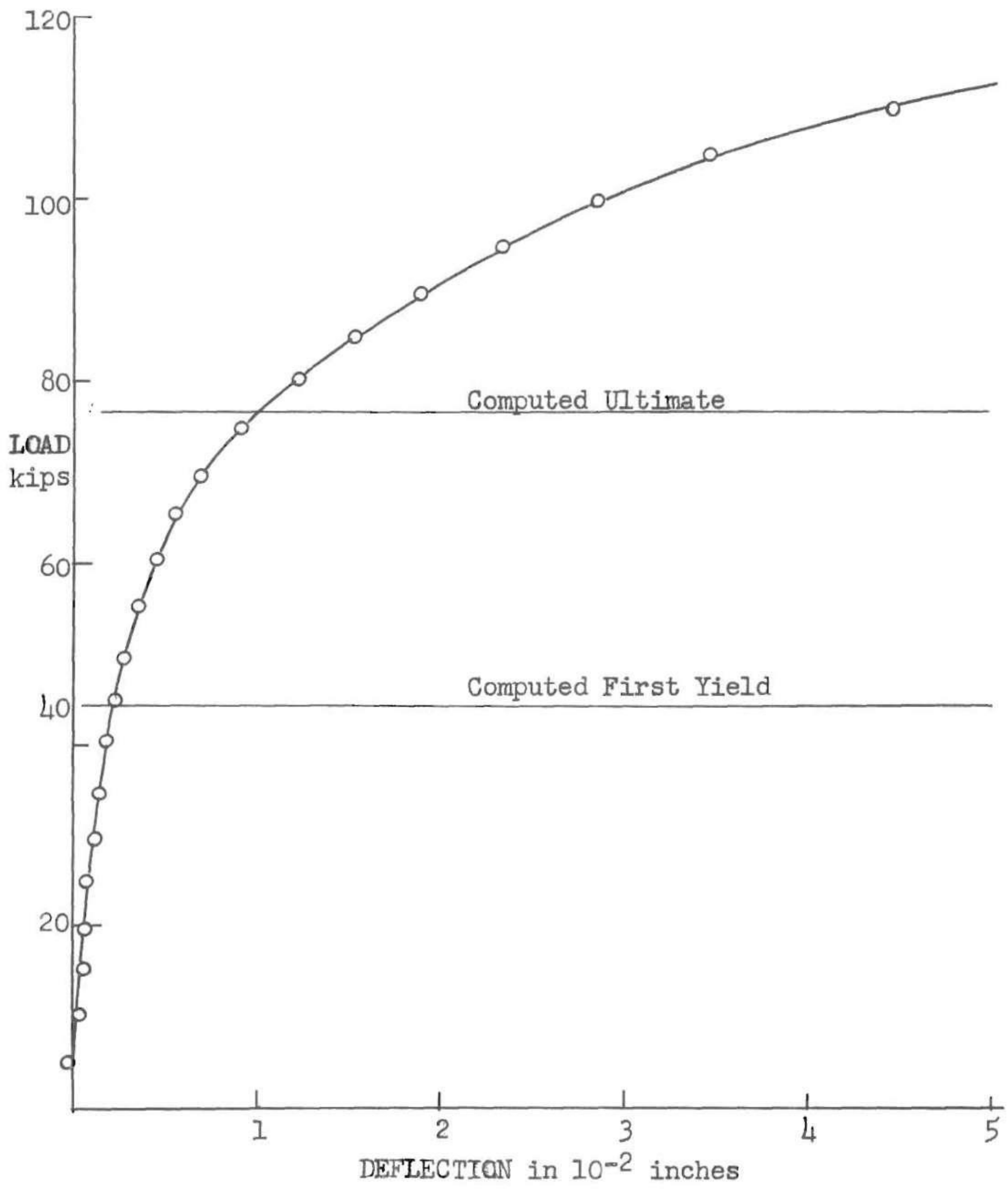


Fig. 12. Deflection of Flange at Web - Test T-5

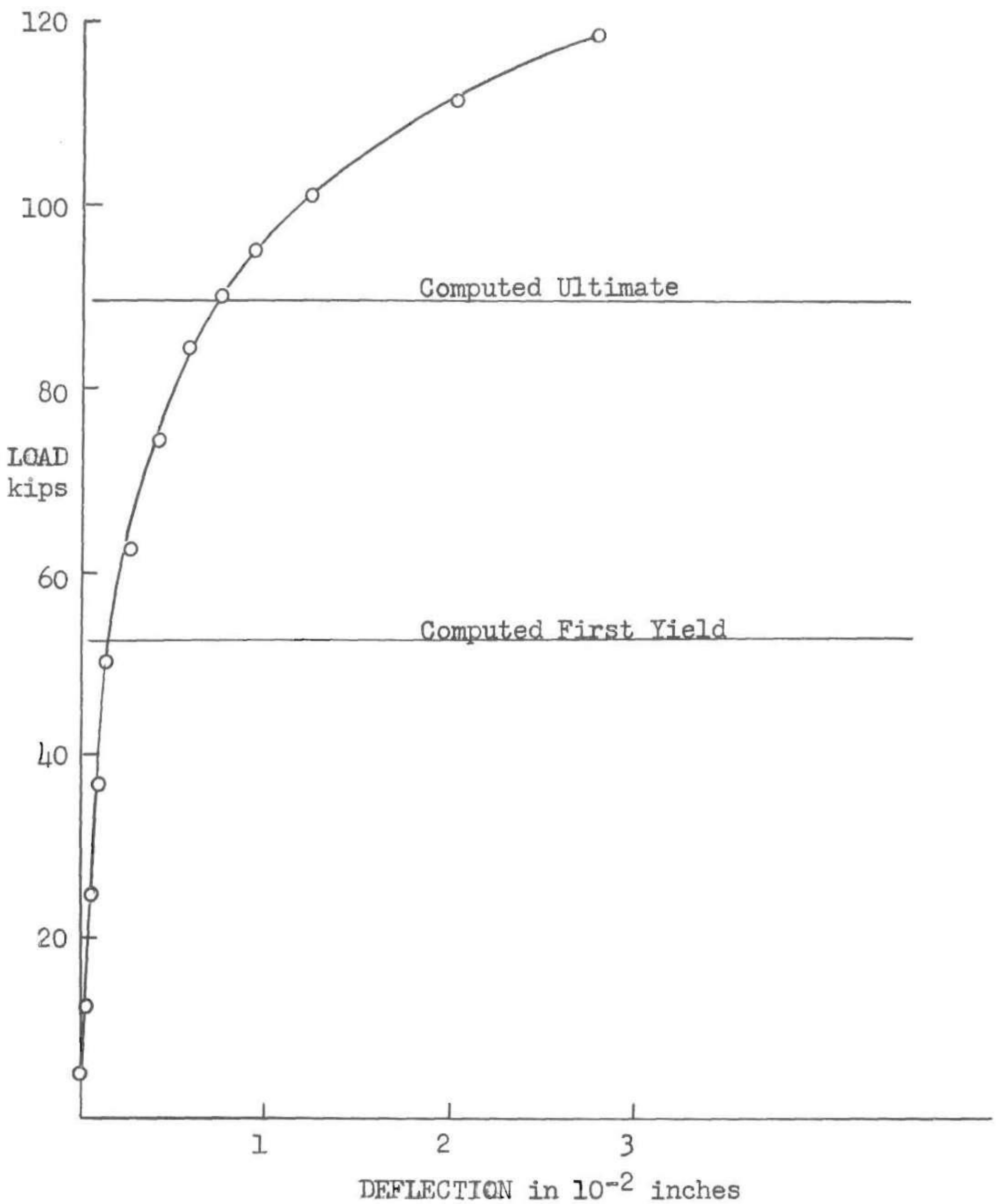


Fig. 13. Deflection of Flange at Web - Test T-6

Table 3. Values For Plotting Predicted Bolt Tension Curves

Test	k	M_p (kip-in)	P_y	T_y	P_u	T_u	P	T_{100}
				(P and T are in kips)				
(Equation) used				(8a)	(4b)	(8b)		(8c)
T-1	0.800	62.2	53.8	--	90.6	--	--	--
T-2	0.789	48.2	41.0	--	69.7	--	--	--
T-3	0.786	54.2	46.0	19.6	78.4	31.8	100	40.6
T-4	0.786	51.7	43.9	23.0	74.8	36.8	100	49.2
T-5	0.765	53.6	44.3	12.6	76.6	20.6	100	26.9
T-6	0.786	61.9	52.6	22.4	89.6	36.3	100	40.6

Table 4. Bolt Tension in the Bracket Test

Item	Tension at Working Load S.F. = 2 in kips (Avg. of two)	Tension at Predicted Ultimate in kips (Avg. of two)	Largest Tension Sustained in the Test in kips (Avg. of two)	Permanent Set in 10^{-4} in.
B-1	32.3	29.0	37.0	None
B-2	32.3	29.0	37.0	7
B-3	31.0	28.5	36.3	None
B-4	31.0	28.5	36.3	6
B-5	31.6	33.0	46.5	42
B-6	31.6	33.0	46.5	10
B-7	31.1	34.2	68.0	8
B-8	31.1	34.2	68.0	43

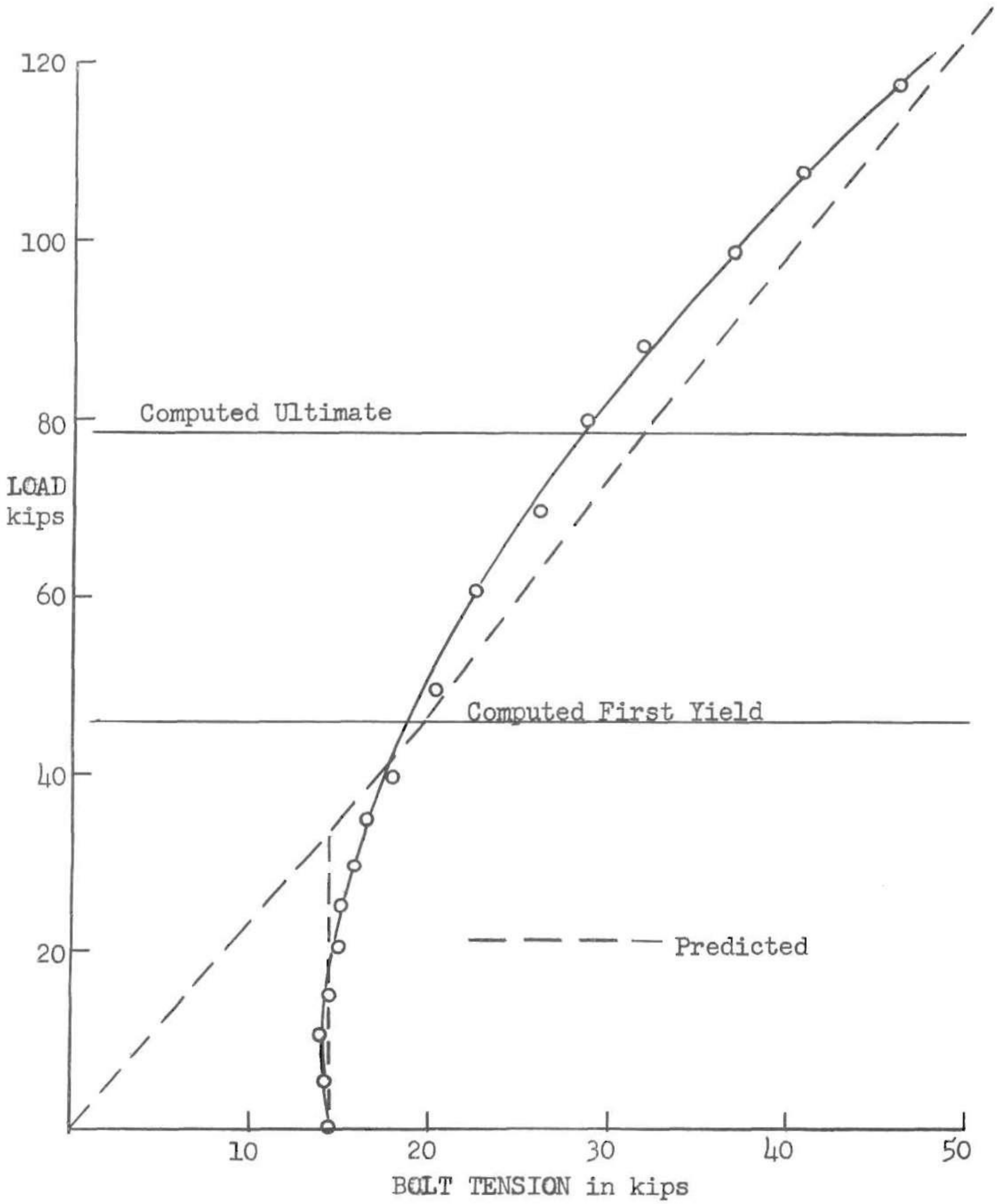


Fig. 14. Computed Bolt Tension Compared with Actual - Test T-3

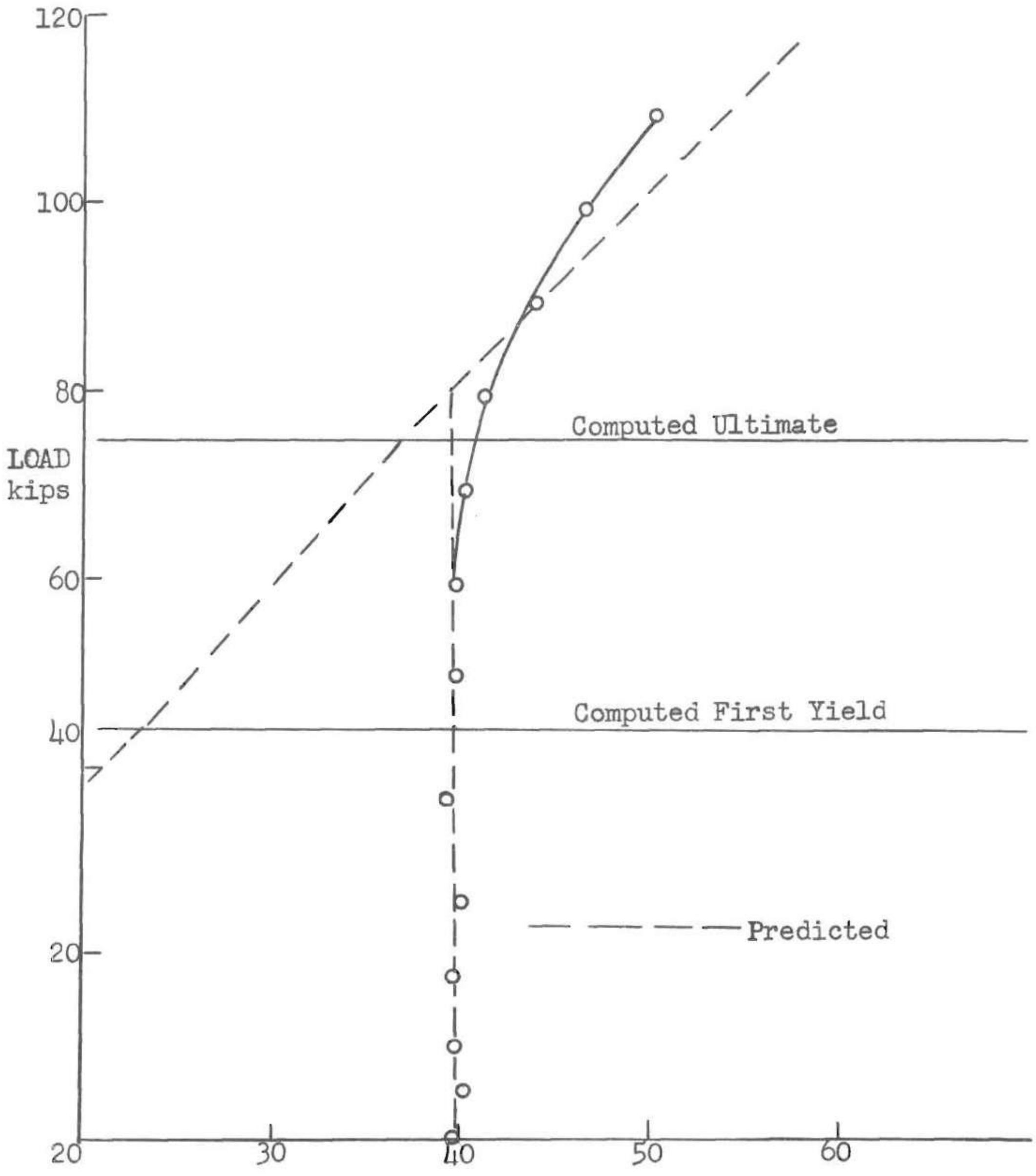


Fig. 15. Computed Bolt Tension Compared with Actual - Test T-4

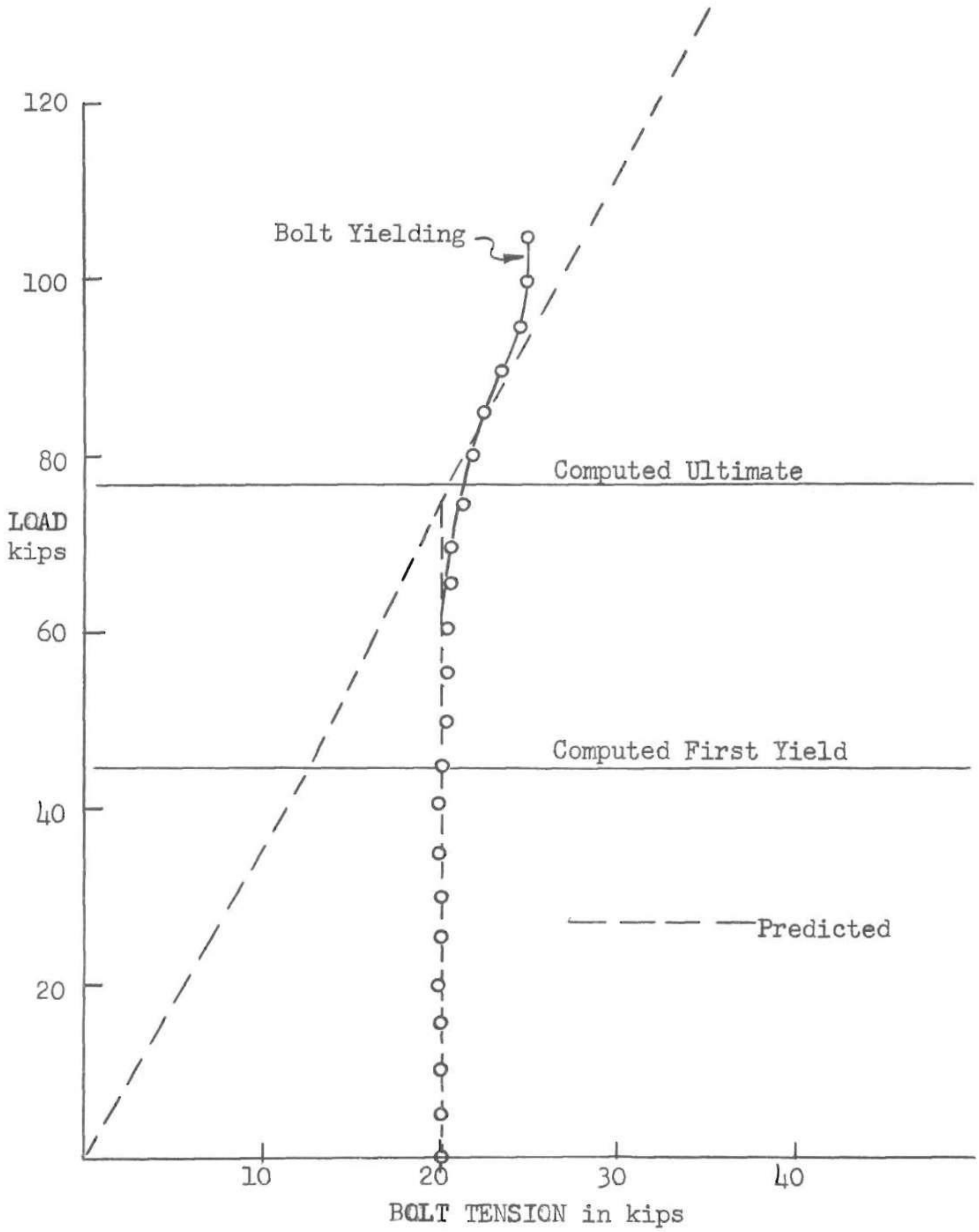


Fig. 16. Computed Bolt Tension Compared with Actual - Test T-5

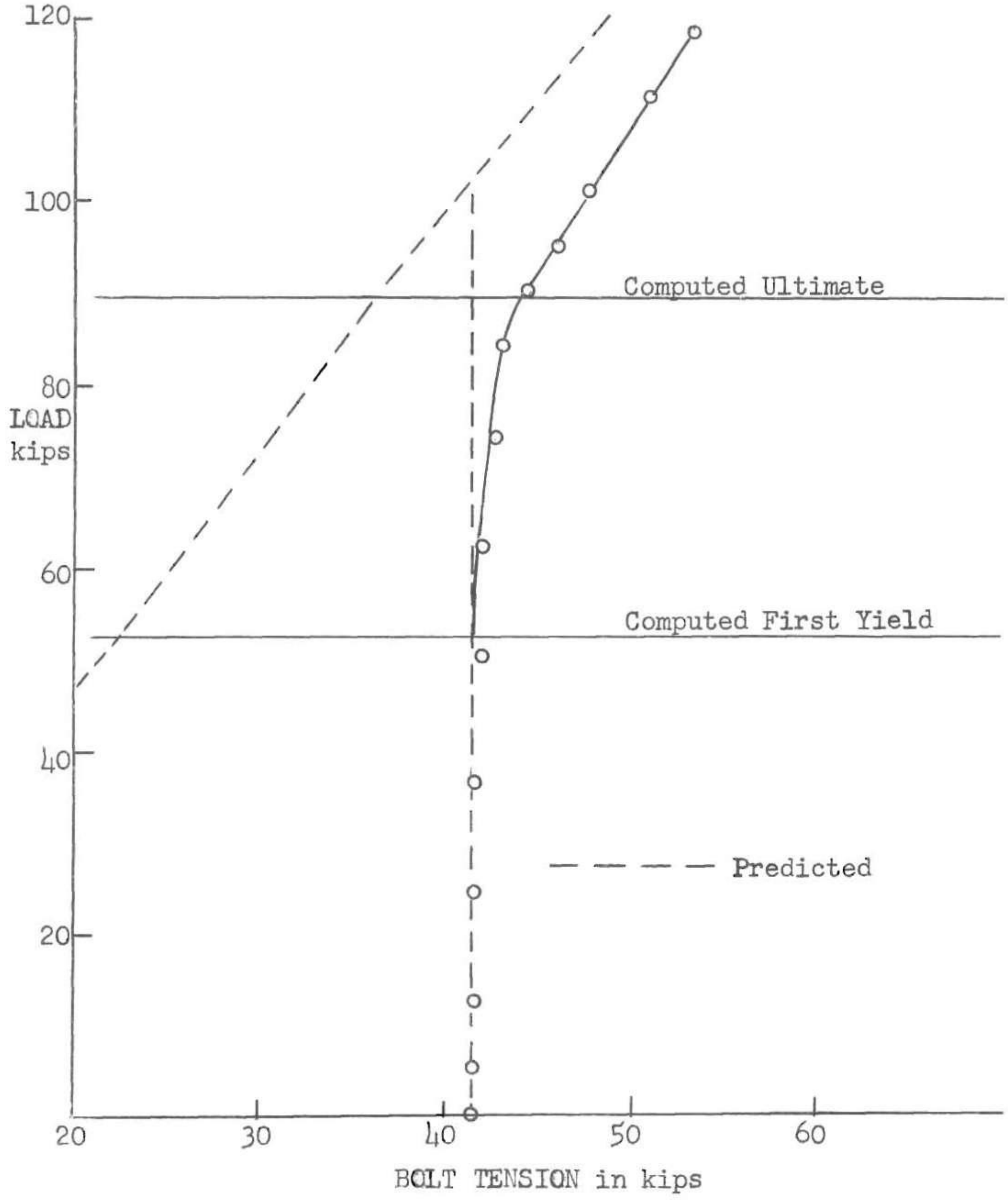


Fig. 17. Computed Bolt Tension Compared with Actual - Test T-6

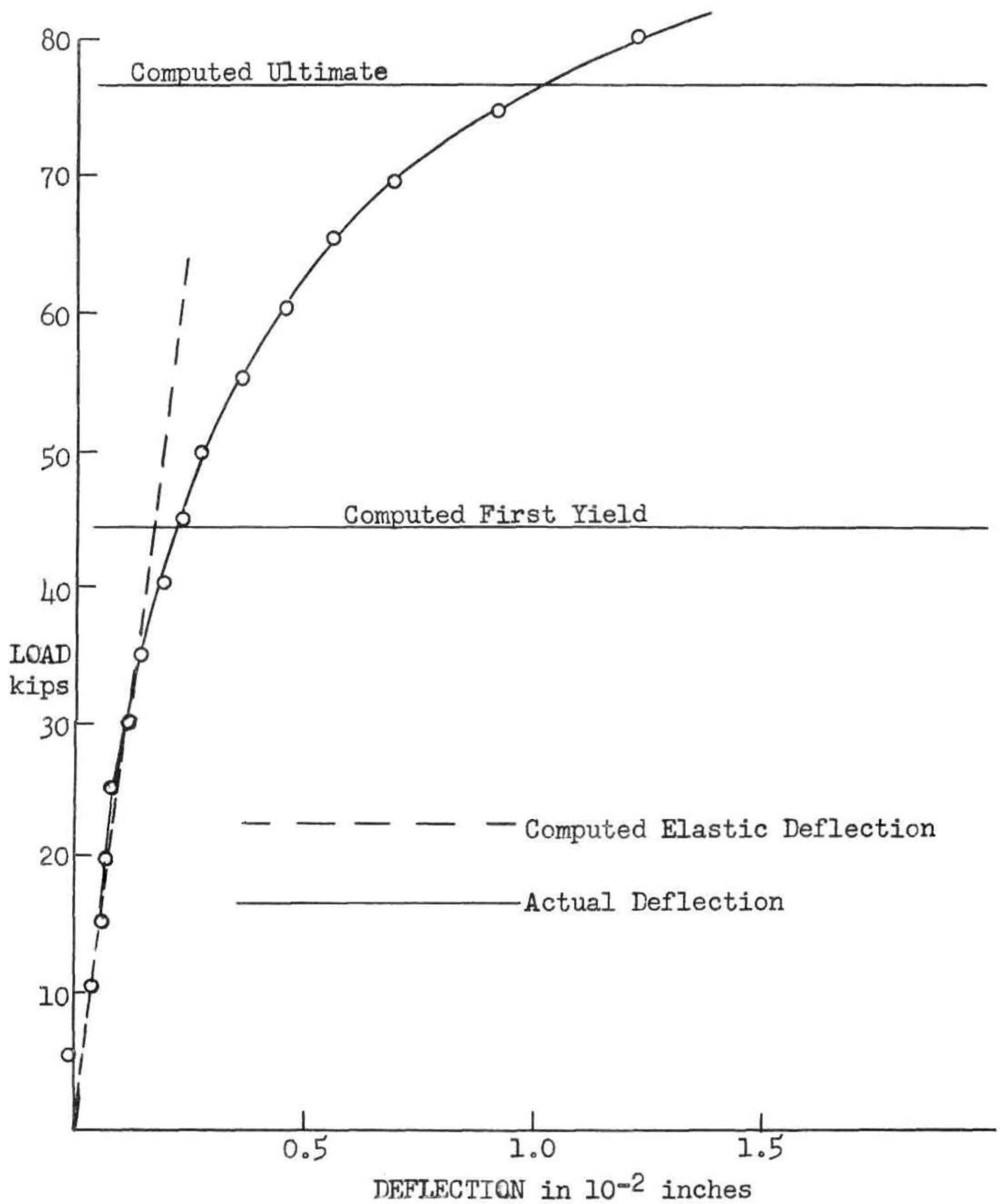


Fig. 18. Comparison of Theoretical and Actual Deflection at Web - Test T-5

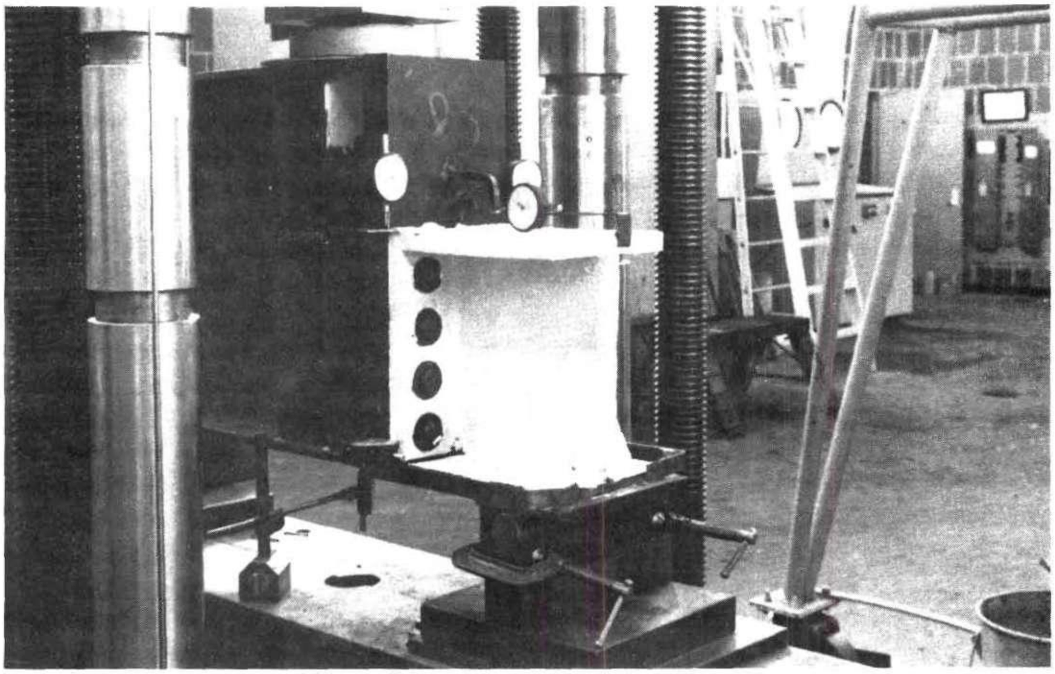


Fig. 19. Bracket Before Testing

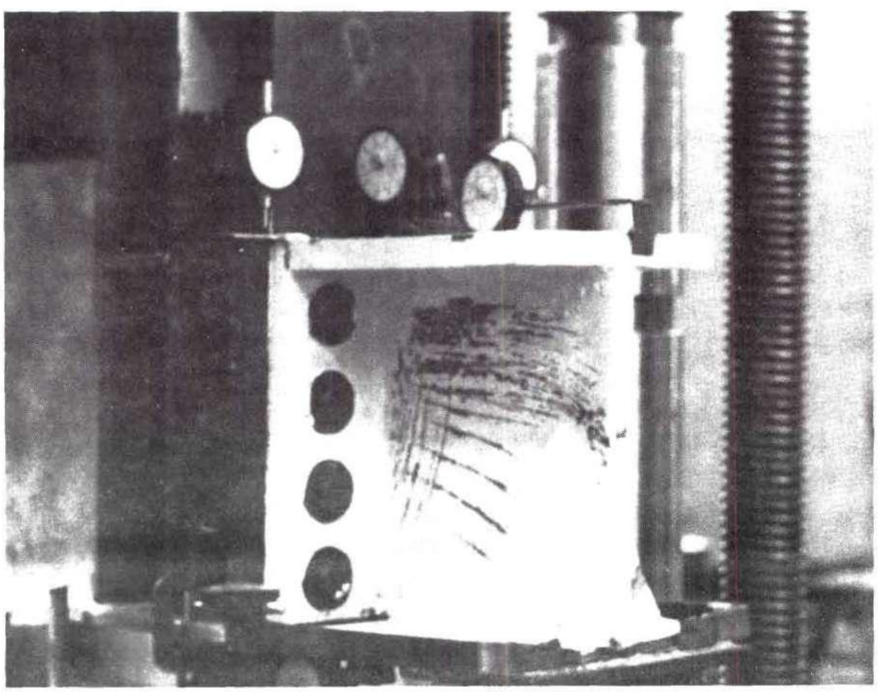
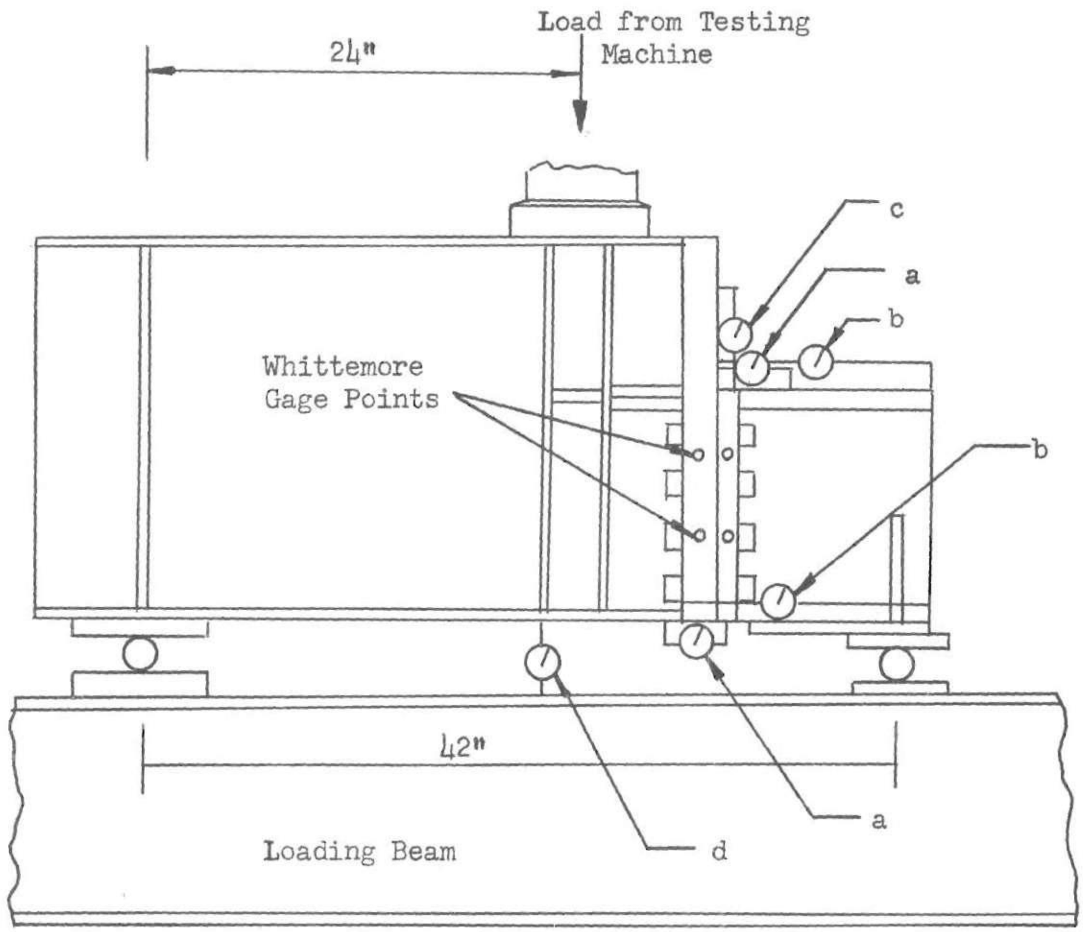


Fig. 20. Bracket at 67 Per Cent Above Ultimate



a. Flange Rotation Dials
b. Load Point Rotation Dials
c. Slip Dials
d. Deflection under Primary Load Point Dial

Fig. 21. Diagram of Bracket Test Arrangement and Instrumentation

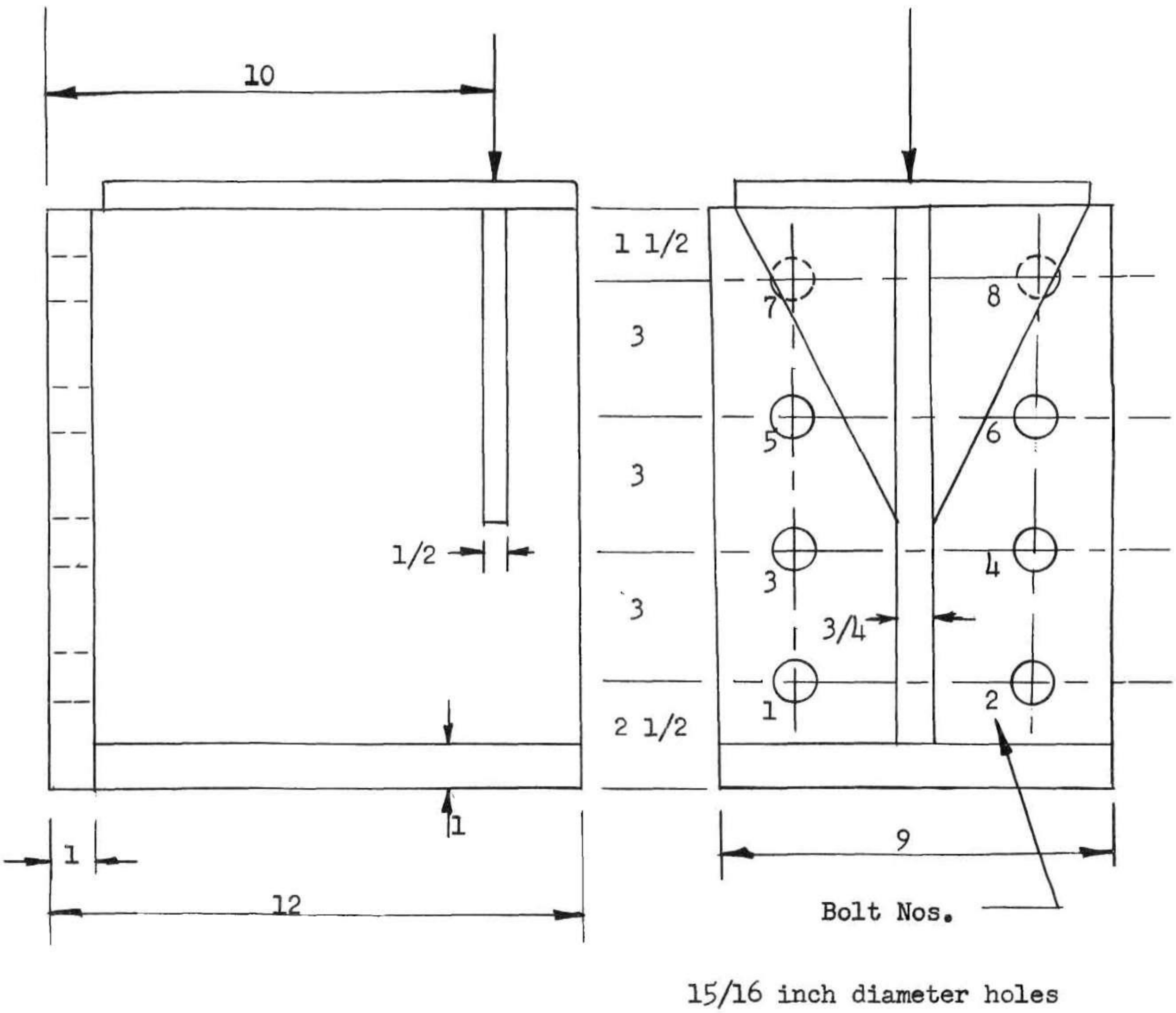


Fig. 22. Hole Arrangement on Bracket

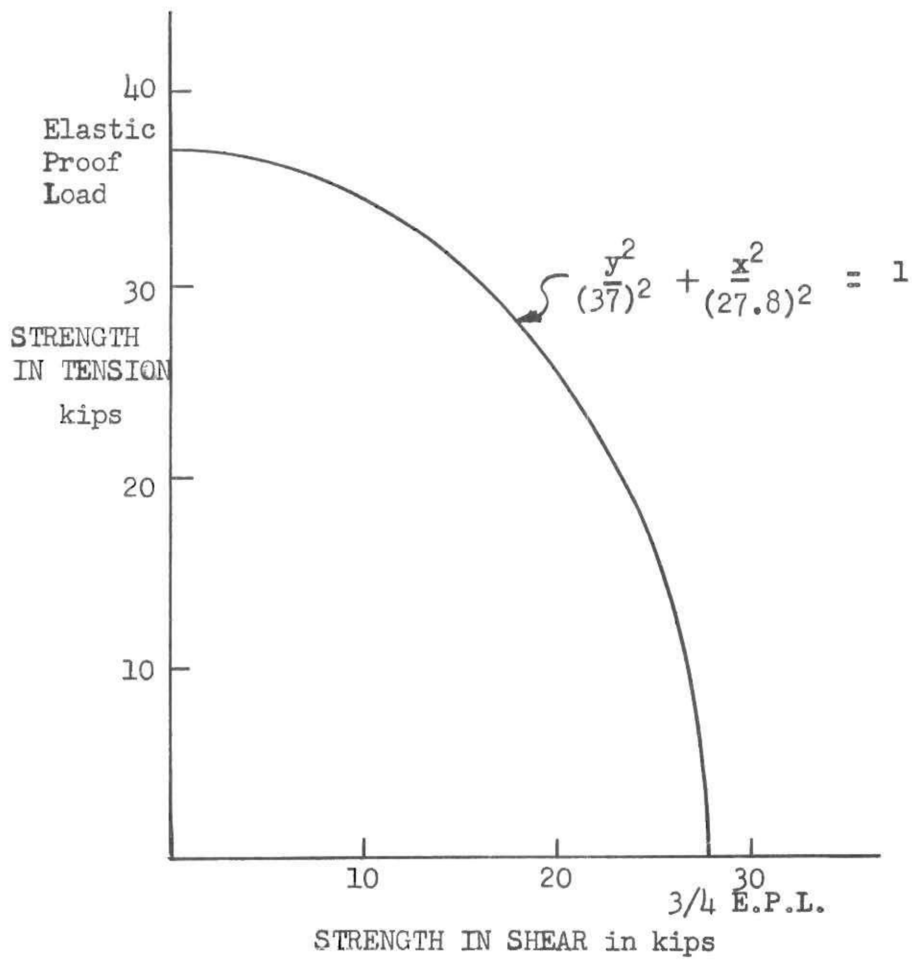


Fig. 23. Interaction Curve for 7/8 inch diameter High Tensile Bolts(Proposed)

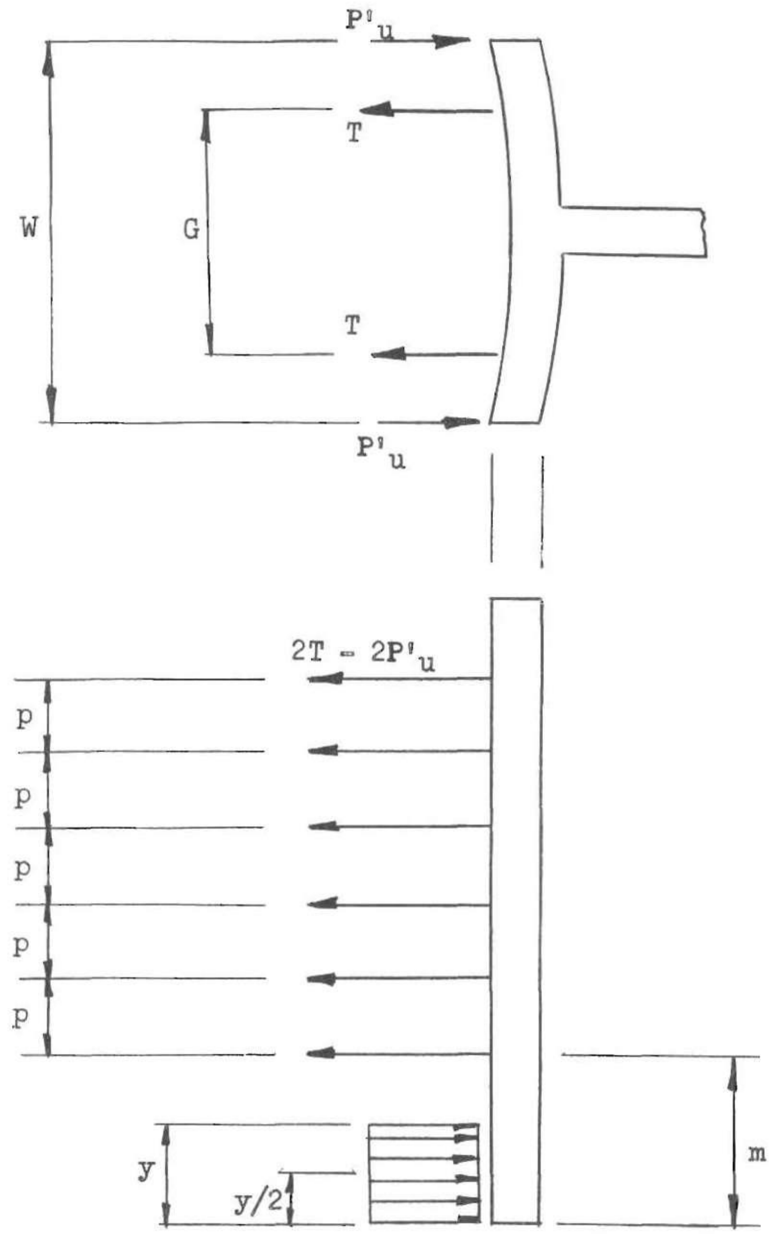
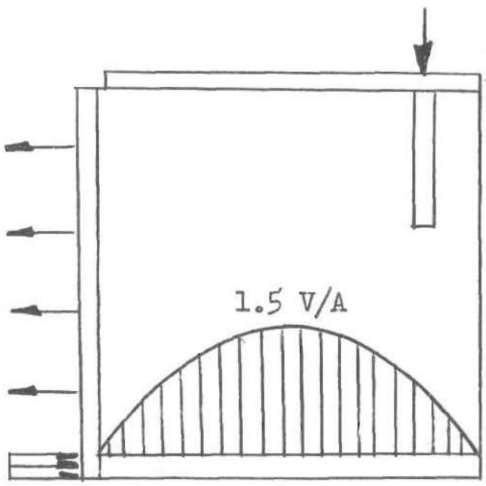
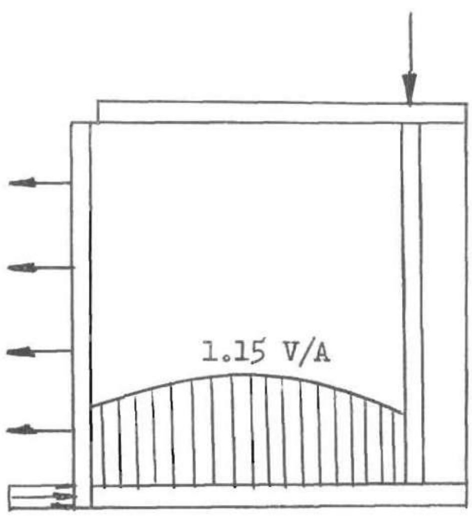


Fig. 24. Ultimate Forces Acting on Flange Plate of a Bracket



Case I - Stiffener Does Not Extend to the Bottom



Case II - Stiffener Extends to the Bottom

Fig. 25. Shear Stress Distribution at Bottom of the Web

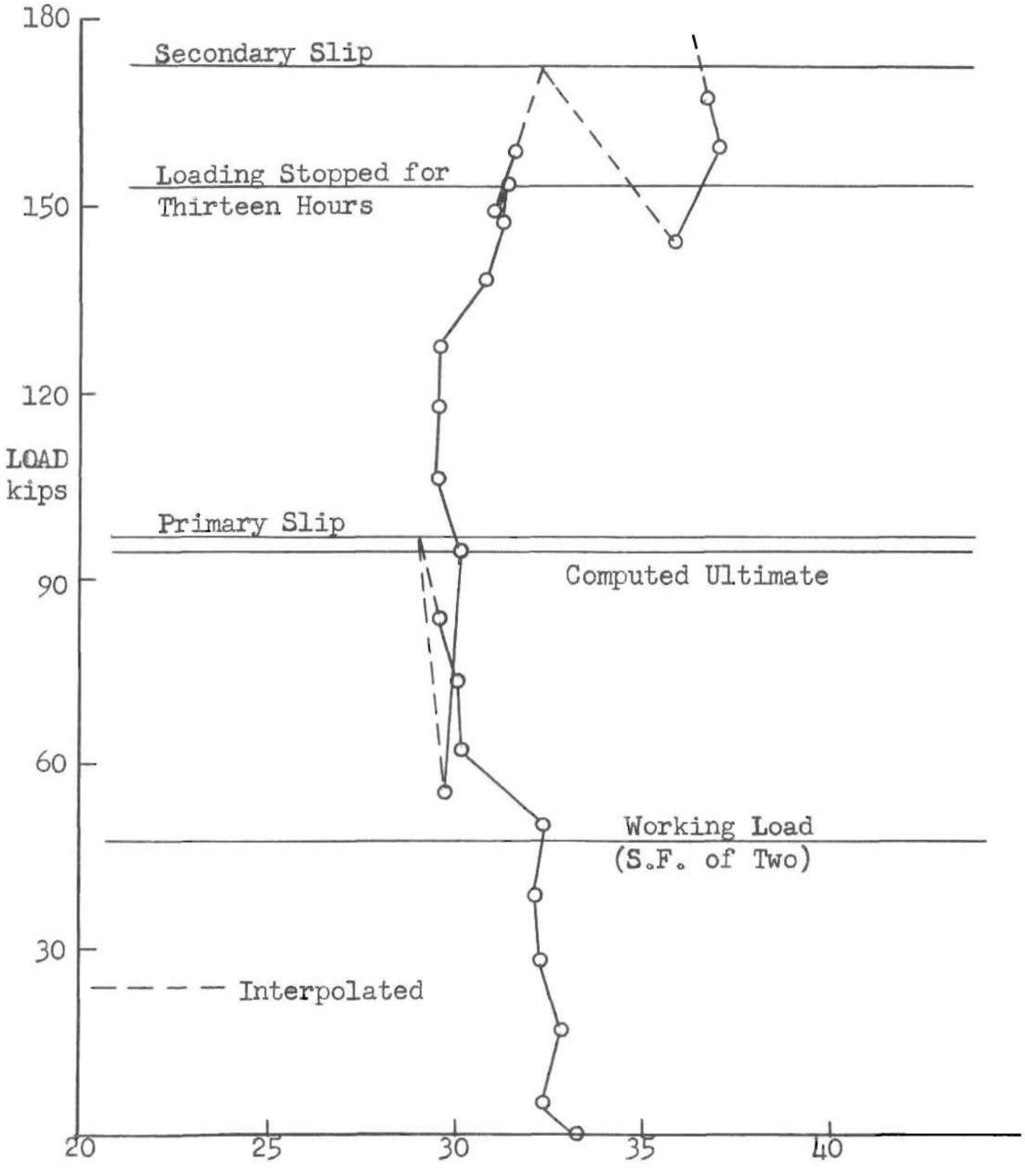


Fig. 26. Bracket Load versus Tension in Bolts 1 and 2

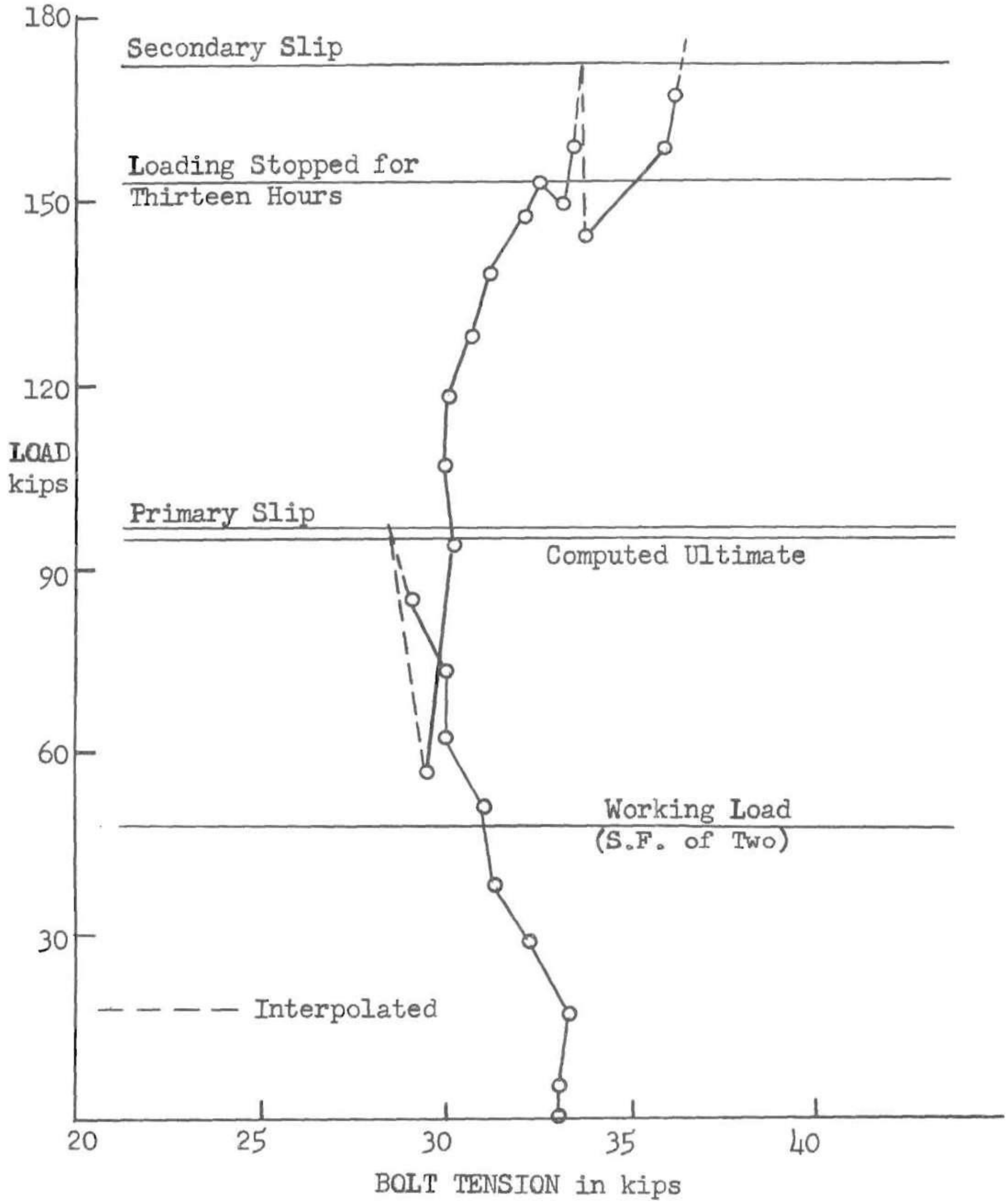


Fig. 27. Bracket Load versus Tension in Bolts 3 and 4

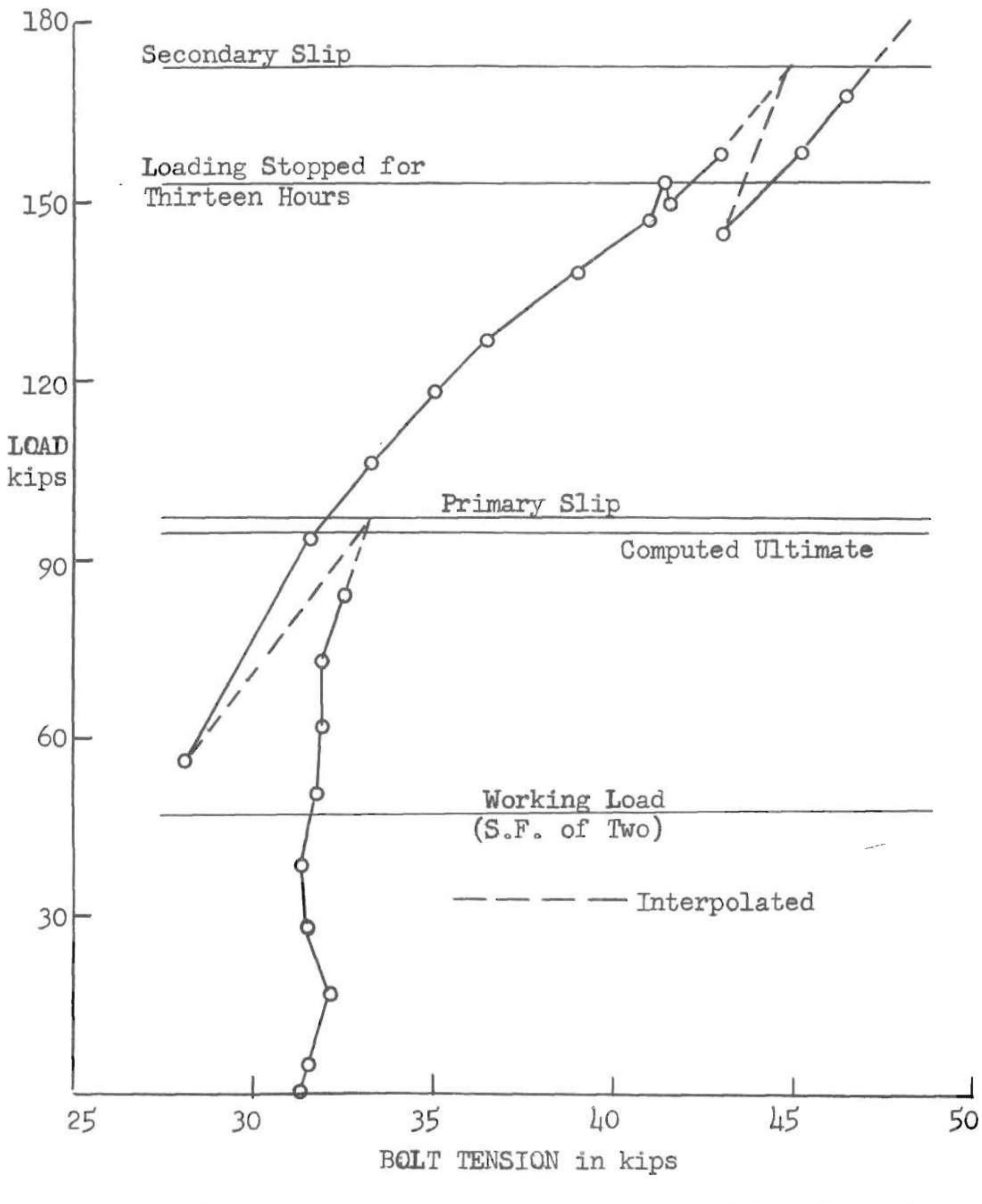


Fig. 28. Bracket Load versus Tension in Bolts 5 and 6

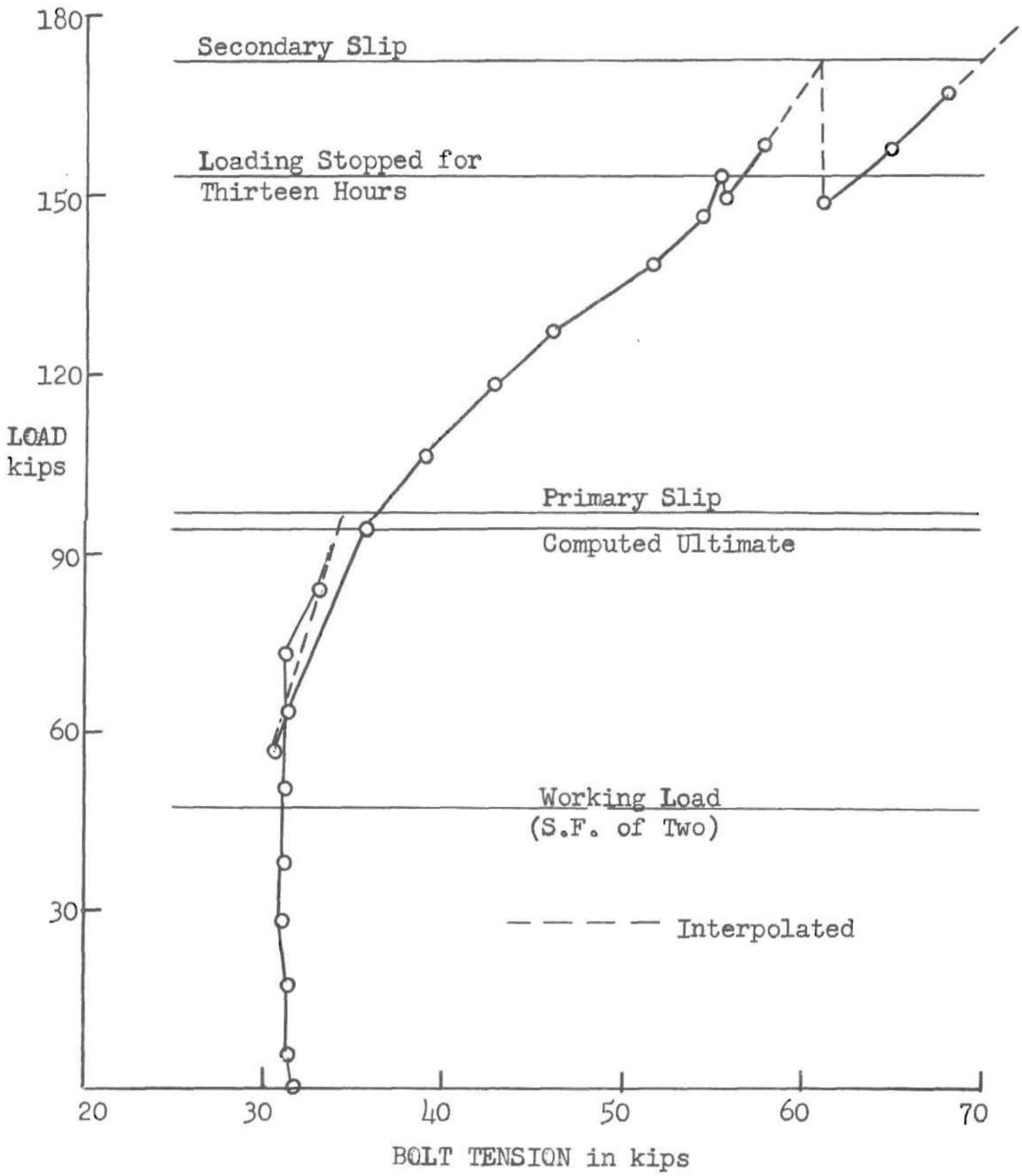


Fig. 29. Bracket Load versus Tension in Bolts 7 and 8

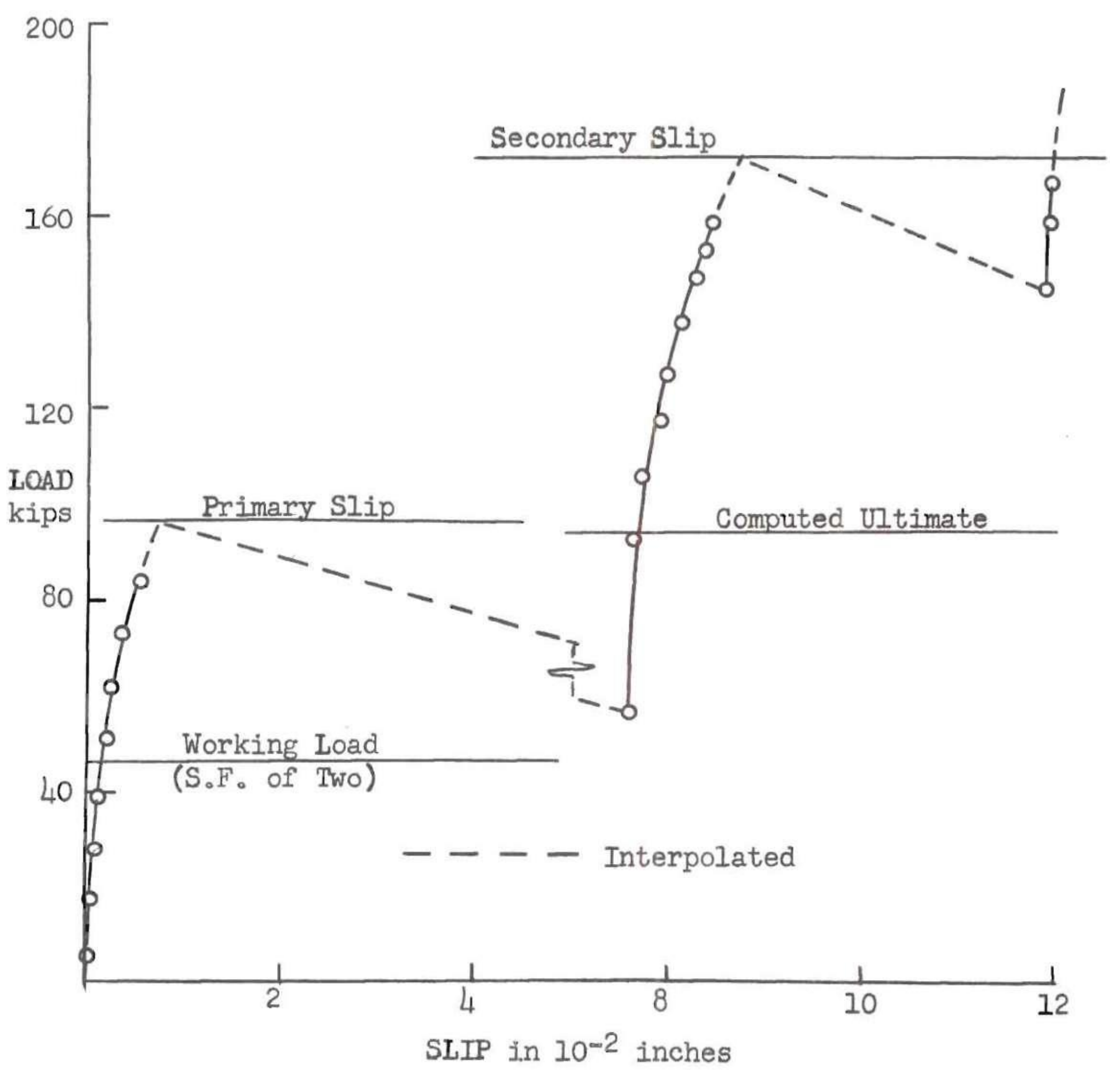


Fig. 30. Average Slip of Bracket

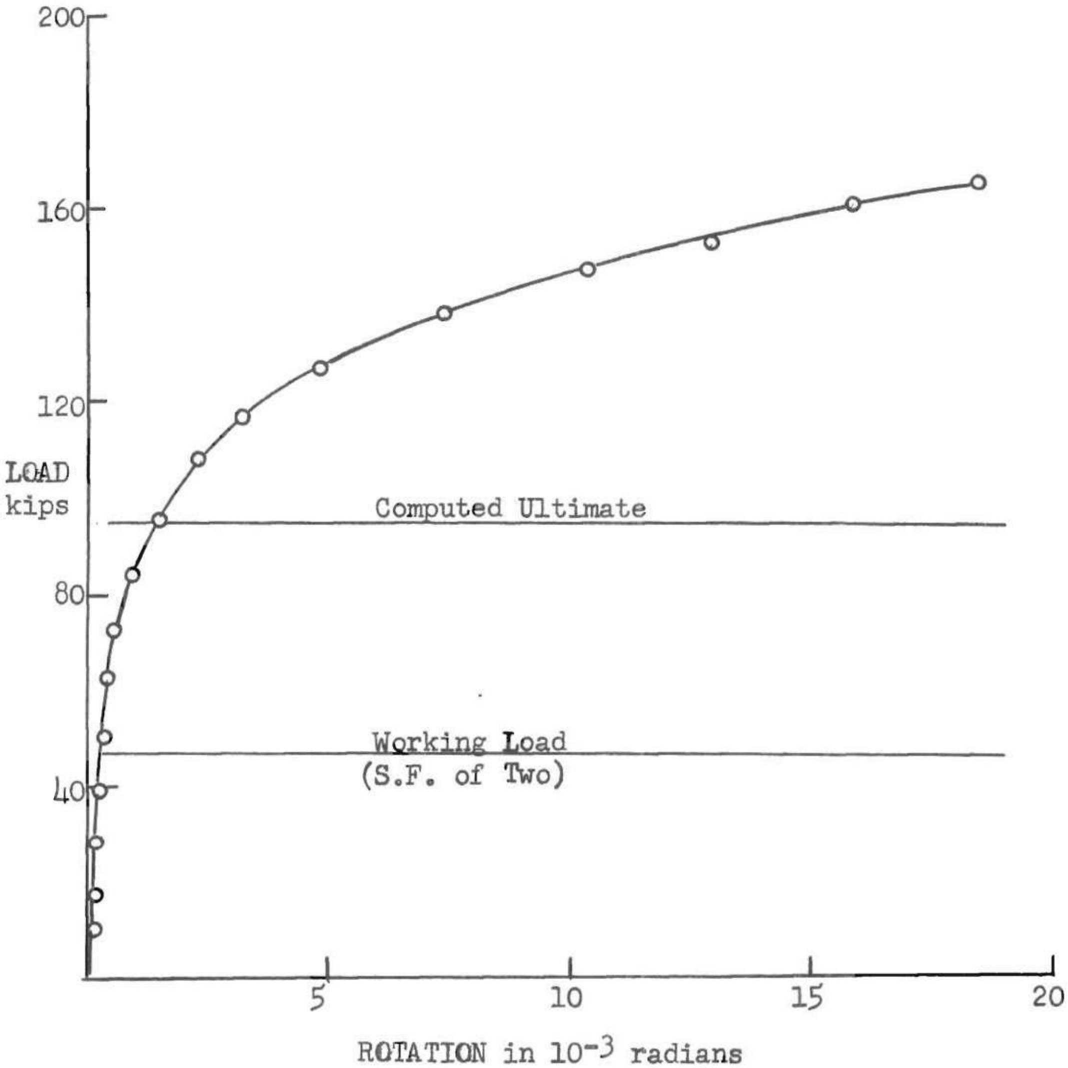


Fig. 31. Rotation of Bracket Flange at the Web

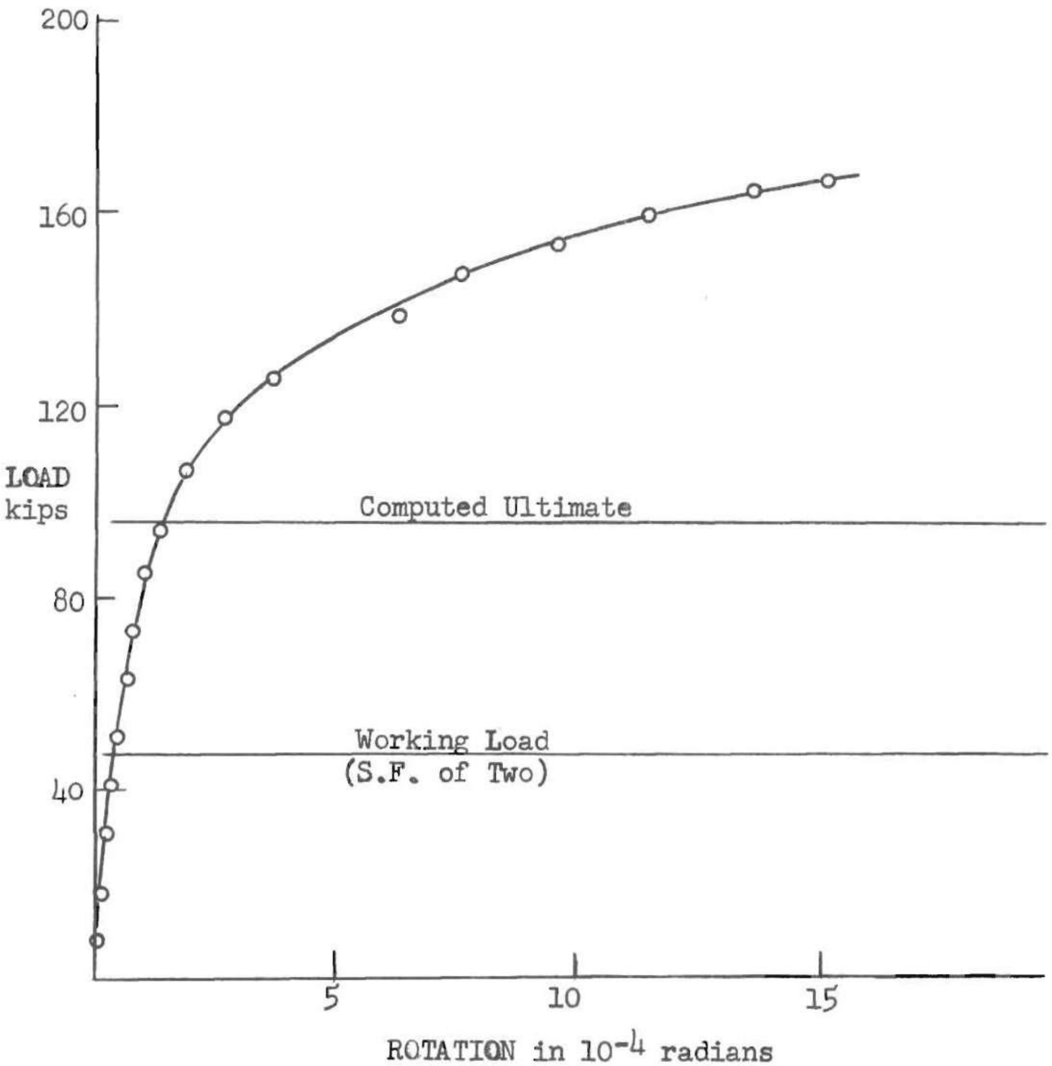


Fig. 32. Rotation of Bracket Load Point

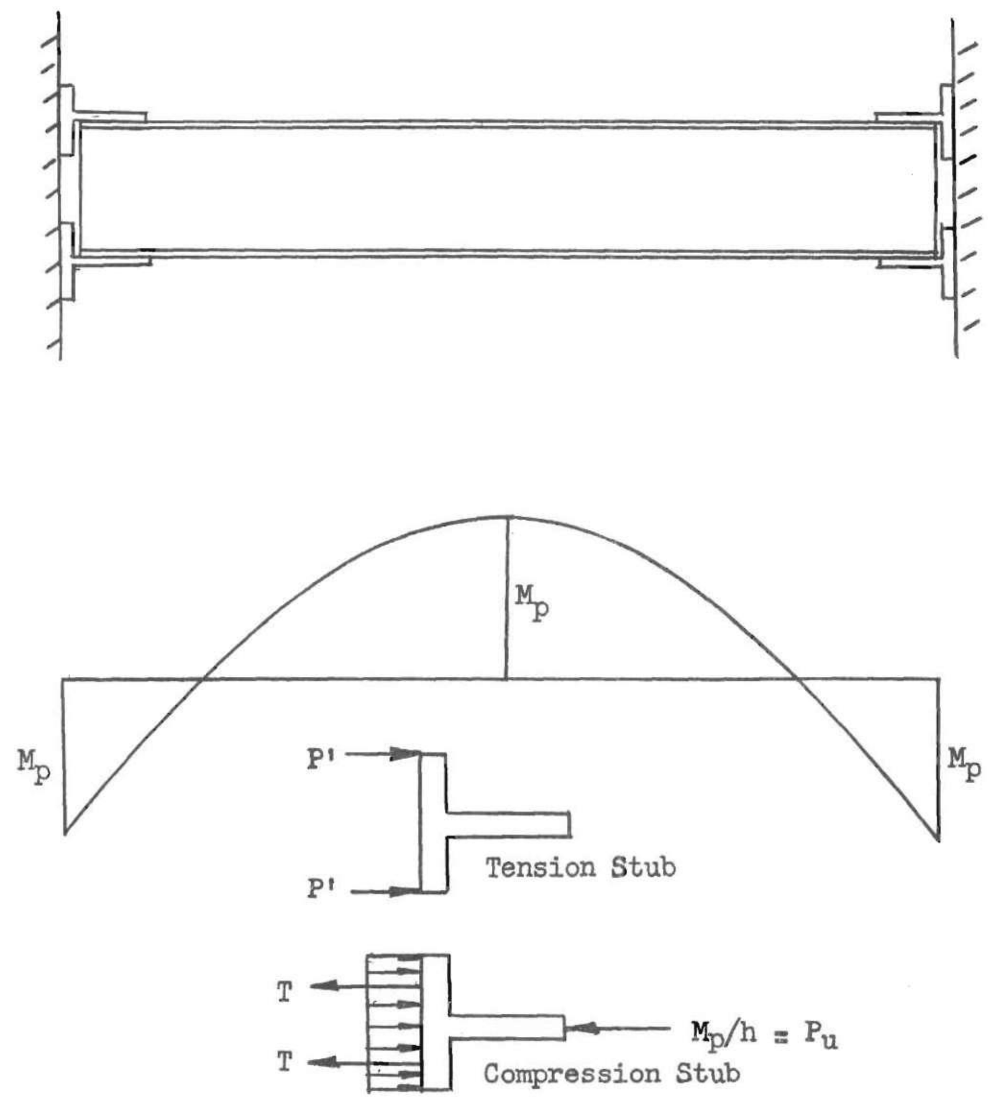


Fig. 33. A Tee Stub Moment Connection

B I B L I O G R A P H Y

BIBLIOGRAPHY

(1) Specifications for Assembly of Structural Joints Using High Strength Bolts, New York, New York: American Institute of Steel Construction, 1954.

(2) Munse, W. H. and Cox, C. L., The Static Strength of Rivets Subjected to Combined Tension and Shear, University of Illinois Bulletin 437, Urbana, Illinois, December, 1956.

(3) Quenched and Tempered Steel Bolts and Studs(A325-55T), Philadelphia, Pennsylvania: American Society for Testing Materials, 1955.

(4) Fuller, J. R. and Munse, W. H., Laboratory Tests of Structural Joints with Over-Stressed High Tensile Steel Bolts, University of Illinois Experiment Station Progress Report of Project IV, Urbana, Illinois, January, 1955.

(5) Square and Hexagon Bolts and Nuts,(ASA B18.2-1952), New York, New York: The American Society of Mechanical Engineers, 1952.

(6) Steel Construction Manual, Fifth edition, New York, New York: American Institute of Steel Construction, 1955.